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RIGID AND FLEXIBLE PAVEMENT AIRCRAFT TIE-DOWNS

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14. ABSTRACT
This report details a three phase project focusing on aircraft tie-downs, which describe mechanisms designed to secure aircraft to the pavement surface. The objective of this project was to determine the pull-out capacity of existing tie-downs and to develop alternative lightweight and heavyweight aircraft tie-downs. The Air Force Civil Engineering Support Agency (AFCESA) categorizes lightweight tie-downs as tie-downs with a vertical pull-out resistance of 17,000 pounds and heavyweight tie-downs as tie-downs with a vertical pull-out resistance of 37,700 pounds. Phase 1 entailed determining the pull-out capacity of existing shepherd’s hook tie-downs in rigid and flexible pavements. Phase 2 and 3 testing focused lightweight and one heavyweight tie-down. Phase 3 testing resulted in the development three possible lightweight tie-downs, although additional testing is necessary before recommending this option.

15. SUBJECT TERMS
rigid pavement aircraft tie-downs, flexible pavement aircraft tie-downs, aircraft mooring points, aircraft anchoring systems

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1. EXECUTIVE SUMMARY

The Air Force Civil Engineer Support Agency (AFCESA) tasked the Air Force Research Laboratory (AFRL) with a three-phase project focusing on aircraft mooring points. The project consisted of the following phases: 1) determining the pull-out capacity of existing anchoring systems in both rigid and flexible pavements; 2) developing new rigid pavement aircraft anchoring systems; and 3) developing new flexible pavement aircraft anchoring systems. The objective was to characterize and develop lightweight and heavyweight anchoring systems. Anchoring systems, also referred to as tie-downs and mooring points, describe mechanisms designed to secure aircraft to the pavement surface. AFCESA categorizes lightweight anchoring systems as mooring points with a vertical pull-out resistance of 17,000 lbs, and heavyweight anchoring systems as mooring points with a vertical pull-out resistance of 37,700 lbs.

AFRL developed an anchor pulling device to perform the testing. The anchor pulling mechanism was essentially a load cell attached to a hydraulic-powered ram. Load cell data were transmitted to a data acquisition system, which allowed testing personnel to conduct anchor testing and quantify the pull-out resistance of various anchoring systems.

Phase 1 testing was conducted at Eglin Air Force Base (AFB), FL. Existing shepherd’s hook anchors, located in both rigid and flexible pavement sections of the airfield, were load tested to determine pull-out capacity.

Phase 2 testing was conducted at the AFRL test site on the Aircraft Operating Surface (AOS) Portland cement concrete (PCC) test pad. This phase of testing focused on developing rigid pavement mooring points capable of meeting the heavyweight and/or lightweight threshold. Two separate tie-downs were installed and load tested.

Phase 3 testing focused on developing flexible pavement mooring points capable of meeting the heavyweight and/or lightweight threshold. This phase of testing was conducted at three locations, each with a distinct soil profile. Several tie-downs were installed and load tested at each location to develop a performance matrix based on soil conditions.

Phase 2 testing resulted in the development of one heavyweight and two lightweight anchoring systems. Phase 3 testing resulted in the development of three possible lightweight anchoring systems. Further flexible pavement tie-down testing is necessary before recommending the three possible anchoring systems to serve as lightweight aircraft mooring points. The additional testing should focus on three anchors tested in Phase 3: fully grouted piers, AFRL grouted anchors, and Tri-Talon anchors. Additionally, AFRL recommends installing and load testing two additional flexible pavement mooring points: helical anchors and Sting Ray earth anchors.

2. INTRODUCTION

2.1. Background

Aircraft mooring points are an integral component of airfields. It is necessary to tie aircraft down to a mooring point to ensure that the aircraft remains stationary in the event of various weather phenomena, such as high intensity winds. Additionally, periods of aircraft maintenance and loading require that planes remain stationary. In some instances, tie-downs also serve as a static grounding point. This report focuses on anchoring systems specifically designed to physically secure the military airplane.

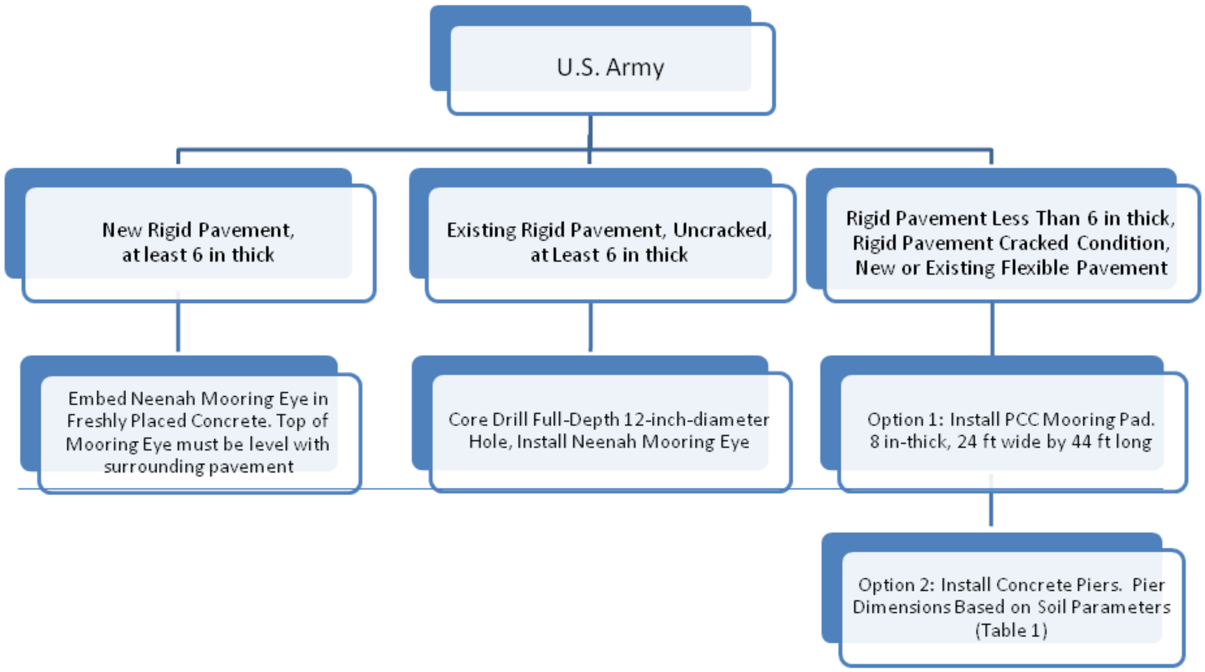
Tie-down and mooring practices were intently examined after severe wind damaged more than 150 Army aircraft at Fort Hood, TX, and Fort Polk, LA, in May and June of 1989, respectively. In response, the U.S. Army Aviation Systems Command issued Technical Manual TM 101520-250-23-1 "General Tie-Down and Mooring on All Series Models AH-64, UH-60, CH-47, UH-1, AH-1, AND OH-58 Helicopters". This technical manual specified that Army aircraft anchoring systems were required to provide an uplift resistance of 20,000 lbs to an applied force in any direction, which was subsequently revised to require an uplift resistance of 15,000 lbs applied at 20.5° relative to the pavement surface[(1).] The current guidelines, available in United Facilities Criteria (UFC) 3-260-01, *Airfield and Heliport Planning and Design*, require construction of Army aircraft aprons to include tie-downs designed to resist a 15,250-lb load applied at a 19.15° angle relative to the pavement surface[(2)].

The U.S. Army and U.S. Air Force service branches employ slightly different physical mooring points and installation layouts according to pavement type and condition. The specifications are designed to meet the loading demands supplied by various aircraft associated with each individual branch. The type of mooring point and layout is dependent upon the pavement type and thickness, and in some instances the condition of the base and sub-base layers.

This report describes testing performed in both rigid and flexible pavement surfaces. Rigid pavement systems consist of PCC of various thicknesses[(3)]. Rigid pavement installation of aircraft tie-downs is typically preferred to flexible pavement installations. This is due to the fact that the concrete matrix often provides the mooring point with sufficient strength to resist pullout.

Flexible pavement systems comprise hot mix asphalt (HMA) layers of various thicknesses. Unlike rigid systems, the asphalt matrix usually does not provide adequate strength in response to the requisite pull out loading demands. Therefore, flexible pavement tie-down installation is typically extremely labor and equipment-intensive in comparison to rigid pavement tie-down installation.

Numerous remediation tactics are utilized to ensure that aircraft mooring points achieve the proper pullout resistance when installed in flexible sections of the airfield. Methods include removing a section of the asphalt and replacing it with a concrete mooring pad, and also the installation of concrete piers into the sub-grade. Figures 1 and 2 detail aircraft tie-down installation guidelines for U.S. Army and U.S. Air Force facilities. Table 1 provides PCC pier dimension guidelines for various soil conditions.



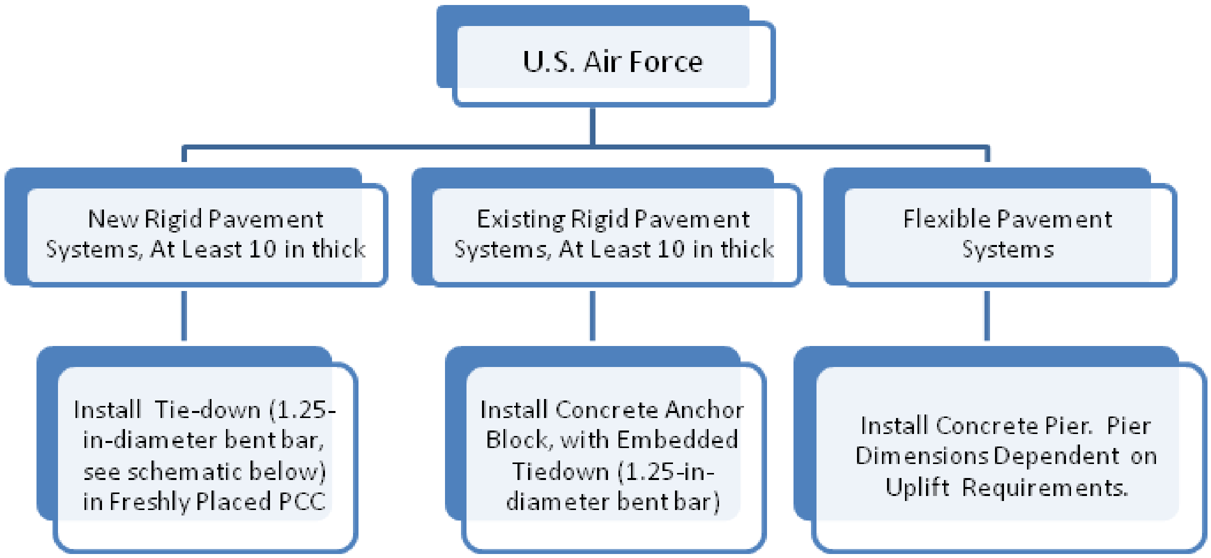
Notes:

1. Existing ¾ -in-diameter, 6-ft-long, shepherd's hook tie-downs are considered adequate if they meet the following conditions:
    a. Installed in rigid pavement
    b. No signs of deformation or corrosion
    c. Rods are inspected for deformation and corrosion once a year and after each storm event with winds greater than 50 knots
2. Existing ¾ -in- diameter, 6-ft-long, shepherd's hook tie-downs are considered inadequate and require replacement if:
    a. Exhibiting signs of deformation or corrosion
    b. Installed in a flexible pavement surface, including those with a PCC block at the surface

Figure 1. U.S. Army Tie-down Guidelines(2)

Table 1. U.S. Army Pier Dimensions for Various Soil Conditions(2)

Cohesive Soils				
Friction Angle Ø (in Degrees)	Pier Diameter		Pier Length	
	mm	ft	mm	ft
Ø < 20°	600	2.0	2,100	7.0
20° < Ø < 30°	600	2.0	1,800	6.0
Ø > 30°	500	1.5	1,800	6.0
Cohesionless Soils				
Unconfined Compressive Strength: (q_u in kg/m^2 [lb/ft^2])	Pier Diameter		Pier Length	
	mm	ft	mm	Ft
q_u < 5,000 kg/m^2 [q_u < 1,000 lb/ft^2]	600	2.0	1,800	6.0
5,000 < q_u < 19,500 kg/m^2 [1,000 < q_u < 4,000 lb/ft^2]	500	1.5	1,800	6.0
q_u >19,500 kg/m^2 [q_u > 4,000 lb/ft^2]	500	1.5	1,200	4.0



Notes:

1. Pier dimensions for flexible pavement systems must be designed to accommodate uplift requirements. For 37,700-lb uplift requirement, minimum pier dimensions are 6 ft by 6 ft by 7 ft (PCC can be assumed to weigh 150 lb/ft^3).

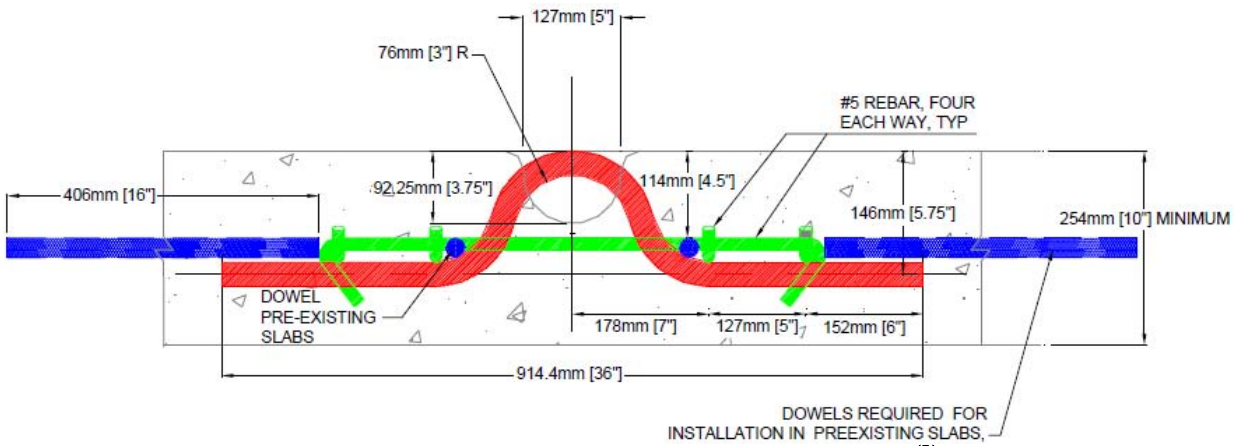


Figure 2. U.S. Air Force Tie-down Guidelines(2)

Many airfields have sections composed of PCC with an asphalt overlay. There is no specific guidance for this scenario, but it is logical to assume that the asphalt section provides a negligible contribution and installation techniques are based on the thickness of the concrete underlay

2.2. Scope

This project consisted of three distinct phases. Phase 1 involved determining the pull-out resistance of mooring points installed in existing rigid and flexible pavement systems. Phases 2 and 3 entailed developing alternative tie-downs for use in rigid and flexible pavements, respectively. This report details each phase of the project.

2.3. Objective

The objective of this project encompassed two major themes: determining the strength of existing anchors and developing alternative tie-down systems capable of meeting the demands supplied by AFCESA for USAF military aircraft.

AFCESA tasked AFRL with developing tie-down anchors capable of meeting lightweight and heavyweight load classifications. Lightweight anchors have a minimal pull-out resistance of 17,000 lbs and heavyweight anchors have a minimal pull-out resistance of 37,700 lbs. Phase 1 involved testing existing anchors to classify them as light and/or heavyweight, while phases 2 and 3 consisted of AFRL developing alternative mooring systems capable of meeting the criteria for one or both of the aforementioned load classifications.

3. ANCHOR PULLING APPARATUS

An anchor pulling mechanism was designed and constructed prior to load testing. The puller, shown in Figure 3, consisted of an Enerpac® hydraulic ram attached to an Omega® load cell. The hydraulic ram was capable of providing 30 t of force and the load cell was rated at 25 t. Hydraulic pressure applied to the ram was regulated with an adjustable valve. A data transfer cable attached directly to the load cell allowed for real time data acquisition in terms of applied force. The load cell generated data readings at a frequency of 10 Hz.

In order to measure deflection a string potentiometer deflection gage was employed. This allowed for real time deflection data that could be correlated to load data at various time intervals. Due to initial equipment limitations, deflection data is only available and provided for the contingency asphalt tie-down section of this report. A shackle attached the load cell (Fig. 4) to the anchor, and the entire apparatus was enclosed in a metal platform to ensure the safety of all testing personnel.

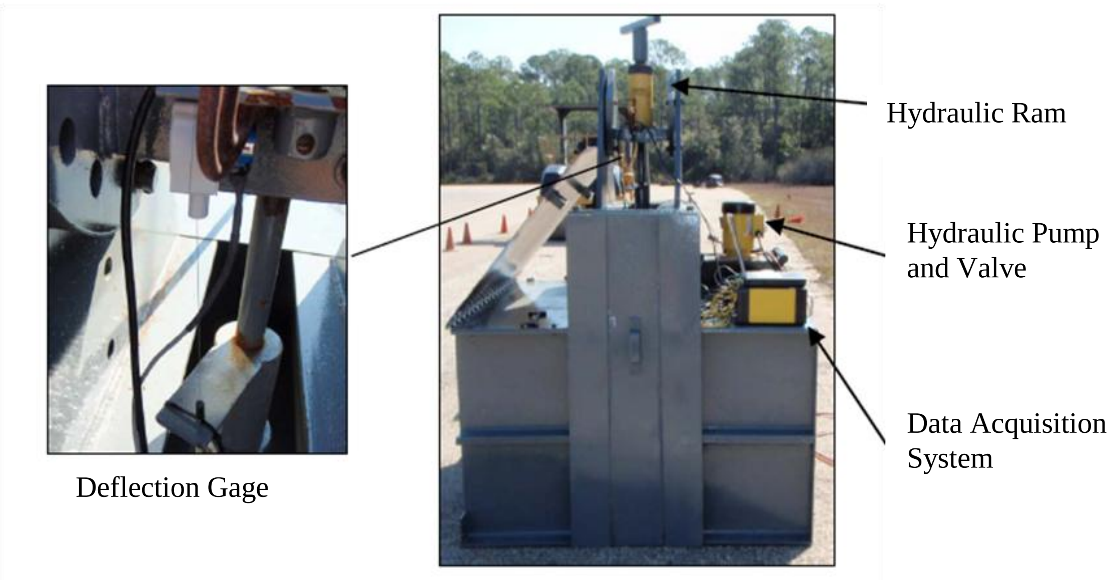


Figure 3. Anchor Pulling Mechanism and Data Components

Design of the anchor puller presented several issues. One of the major issues involved the load rating capacity of the coupling attachments necessary to connect the ram to the load cell and the load cell to the tie-down. It was extremely important that the attachment mechanisms not fail prior to the tie-down. Lifting equipment manufactured by the Crosby Equipment Company was utilized to serve the role of couplers. The major problem regarding this arrangement was that the working load level of the Crosby equipment was well below the load rating of the hydraulic ram and load cell. However, according to Crosby, the ultimate load is four to five times higher than the working load[(4)]. Therefore, the ultimate load rating of the lifting and coupling attachments well exceeded the load rating of both the hydraulic ram and load cell. While not an ideal set-up, the size of higher rated attachment equipment would have required an alternate load cell and hydraulic ram, both of which were deemed undesirable.

Another issue was the angle at which the anchors were pulled. Initially, the anchor pulling apparatus was equipped to provide only a vertical pull, which did not provide an ideal simulation of the load demand supplied by an anchored aircraft. In fact, as previously stated, the Army specifies a pull angle of 19.15° with relationship to the horizontal pavement surface. The angle of pull issue was not remedied until Phase 3 of the testing process and ultimately all testing was conducted using a vertical tensile force.

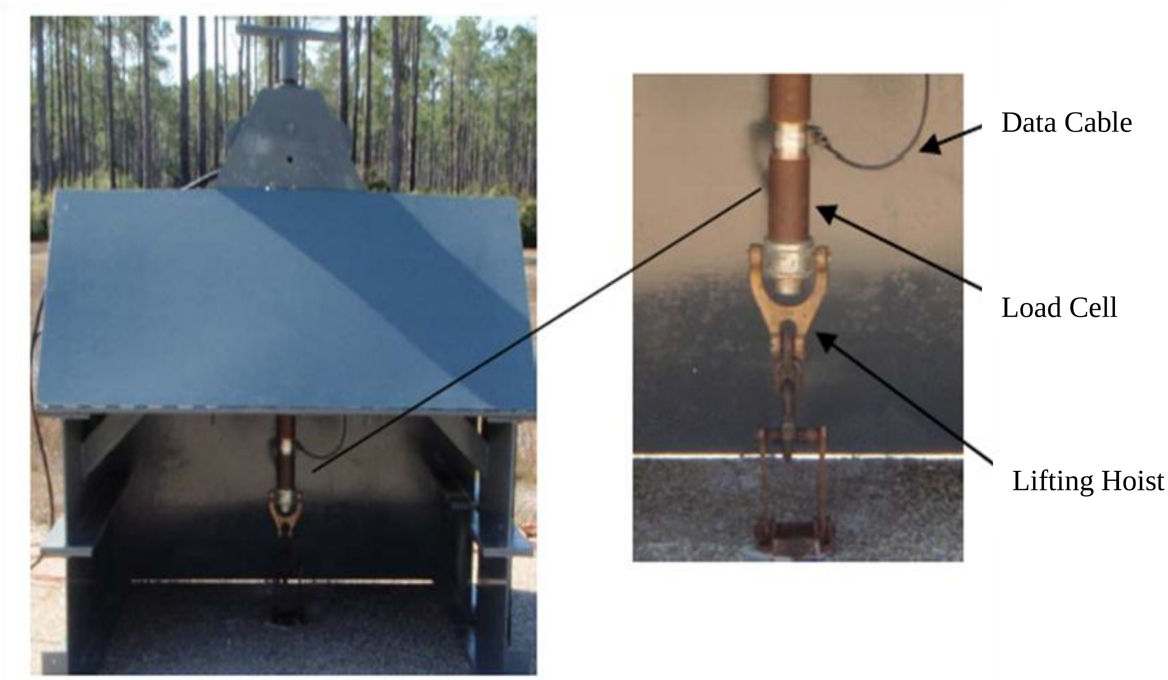


Figure 4. Anchor Testing Components

4. PHASE 1: TESTING EXISTING ANCHORS

One of the primary traditional Air Force tie-downs is a 6-ft x 5/8-in-diameter metal rod, inserted through the pavement layer into the sub-grade[1]. The top of the tie-down is similar to a shepherd's hook, the top of which sits flush with the surface of the pavement layer and serves as an aircraft attachment point. Shepherd's hook tie-downs are typically produced from copper-clad steel, galvanized steel or copper–zinc–silicone alloy. These mooring points have been installed in PCC and asphalt concrete pavements. Importantly, the document UFC 3-260-01 [2] specifically states that these tie-downs are not intended to resist the uplift force of a moored aircraft. However, due to the extensive presence of this particular anchor type in existing airfields, it was necessary to test existing shepherd's hook mooring points and quantify their pullout resistance.

Phase 1 testing was conducted at Eglin AFB, FL. A total of eight of the traditional shepherd's hook anchors were tested to determine their ultimate load capacity. Six of the tie-downs were located in rigid sections of the runway, and the remaining two mooring points were located in flexible pavement sections. Deflection data were not collected during testing, and thus it was not possible to correlate deflection readings with accompanying load data. Therefore, only the ultimate load capacity is provided.

4.1. Rigid Pavement Tie-downs

Six shepherd's hook anchors located in rigid pavement sections were subjected to a vertical tensile force in order to determine the pull-out capacity of each anchor. Each tie-down was located in sections of PCC exhibiting no visible damage. Slab thickness was not known.

Several types of tie-down failures were exhibited during the testing process. In some instances, the entire anchor system and some of the surrounding concrete was partially extracted from the ground. Other failure types included straightening of the metal hook and fracture of the metal hook. Straightening occurred when the end of the hook separated from the compression sleeve, and fracture was exhibited in instances where the hook broke into two pieces. Load cell data graphs for this portion of testing have been provided in Appendix A of this report. Table 2 shows the ultimate load capacity and failure type for each of the rigid pavement shepherd's hook tie-downs tested at Eglin AFB. Figure 5 illustrates the three failure mechanisms.

Table 2. Ultimate Load Capacity and Failure Type of Shepherd's Hook Rigid Pavement Tie-downs

Tie-downs	Ultimate Load (lbs)	Failure Type
1	13,439	Extraction of Anchor from PCC Slab
2	22,530	Extraction of Anchor from PCC Slab
3	21,073	Fracture of Metal Hook
4	16,861	Straightening of Metal Hook
5	25,898	Straightening of Metal Hook
6	11,037	Straightening of Metal Hook

Figure 5. Shepherd's Hook Failure Types

4.2. Flexible Pavement Tie-downs

Two tie-downs were tested in flexible pavement sections of the airfield. The tie-downs were encased in a 12-in square concrete pier of unknown depth (Fig. 6). The shepherd's hook portion of the tie-down was recessed in the concrete pier so that the top of the hook was slightly below the top surface of the concrete pier. Each pier was even with the top of the adjacent asphalt pavement. Test specimens were chosen from pavement sections exhibiting minimal cracking. Some cracking was observed in the asphalt surrounding each pier but the concrete pier and the embedded tie-down appeared undamaged. Data from one test did not transfer from the load cell to the data acquisition system. Thus, results for only one flexible pavement anchoring system are provided in this report. The shepherd's hook tie-down in flexible pavement achieved a pull-out capacity of 12,500 lbs and exhibited a failure mode of partial extraction of the anchor system.

Figure 6. Shepherd's Hook in Flexible Pavement Section—Post-Test

4.3. Phase One: Results and Discussion

4.3.1. Rigid Pavement Tie-downs

The data review allows for limited conclusions. The significant variance in the ultimate load capacity provided by the six shepherd's hook mooring points makes predicting the load rating and ultimate failure capacity of similar anchors difficult at best. Additionally, several failure mechanisms were observed during testing. Failure mechanisms included partial extraction of the anchoring system from the adjacent concrete, fracture of the metal shepherd's hook, and separation of the end of the hook from the compression sleeve. An observation of the data illustrates that even amongst similar failure types there is wide variability in the ultimate load capacity.

Several factors likely contributed to the the test results. No information was available detailing when the various anchors were installed and the type of metal used to construct each tie-down. It is also possible that some, if not all, of the anchors had served as mooring points for several decades. No information was available detailing the concrete type or compressive strength.

Additionally, no data were accessible regarding the number of times each anchor had served as a mooring point, or which type of aircraft had been tethered to the different tie-downs. It is possible that some of the anchors tested had previously been subjected to more rigorous loading demands than others. Metal corrosion likely impacted the different tie-downs to various degrees. Slight variations in the soil profile possibly contributed to anchor performance as well.

A much wider scope of testing is needed to make load capacity predictions with any degree of confidence and accuracy. The alternative is to proof load existing anchors to ensure they are able to meet the requisite loading demands.

4.3.2. Flexible Pavement Tie-downs

Limited conclusions can be drawn from anchor testing performed in the flexible pavement sections at Eglin AFB, FL. Data are available from only one test. A multitude of additional testing is essential to begin the process of adequately predicting the pullout strength of flexible pavement shepherd's hook mooring points.

5. PHASE TWO: CONTINGENCY CONCRETE ANCHORS

Phase 2 testing consisted of developing alternative rigid pavement anchoring systems to the traditional shepherd's hook for contingency environments. This phase of testing also encompassed evaluating the pullout capacity of alternative anchors already being utilized in contingency environments. Testing was performed at the AFRL facility located on Tyndall AFB, FL. The testing site was the AOS's concrete test pad. The various anchors were installed in the test pad and subsequently evaluated to determine loading capacities.

The AOS test pad is sited in a section of PCC 12 in thick. Directly underneath the slab is a 4-in-thick crushed aggregate base course. The subgrade consists of a poorly graded silty sand layer.

Two distinct mooring points, currently present in contingency environments, were selected for testing. These anchors included the Neenah mooring eye and the Hat-Type tie-down. The following sections detail the installation and performance of these two rigid-pavement anchoring systems. Installation timelines and equipment lists are provided in Appendix B of this report.

5.1. Neenah Mooring Eye

The Neenah aircraft mooring point is a commercially available aircraft tie-down. Neenah anchors are currently employed at several military airfields and many civilian airports. They consist of an oval-shaped ductile iron casting with a cross rod to which mooring hooks are attached. According to Neenah Foundry, manufacturer of the Neenah anchor, the cross rod is load rated at 9,000 lbs[5]. Neenah mooring points are installed with and without concrete piers, depending on the depth and condition of the existing concrete pad, as well as the expected load demands.

Grau and Cooksey (1) tested Neenah anchors installed in 6-in and 8-in-thick concrete slabs and determined that Neenah mooring points are able to resist uplift loads in excess of 17,000 lbs. It is imperative to note that the tie-downs tested by Grau and Cooksey were welded to a 10-ft metal grounding rod that had been driven into the ground, and that they were installed before concrete placement. Additionally, tensile force was applied at 20.5° in relation to the pavement surface.

A common field practice involves removing a 12-in-diameter core from the existing concrete slab and placing a Neenah mooring point and fresh concrete in the cored section. This specific installation was not part of the test design. Figure 7 is a Neenah mooring eye, and Figure 8 is a plan and elevation view of an installed Neenah mooring eye.

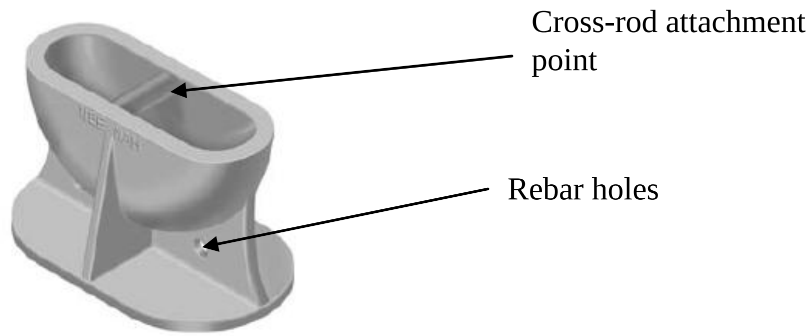


Figure 7. Neenah Mooring Eye[5]

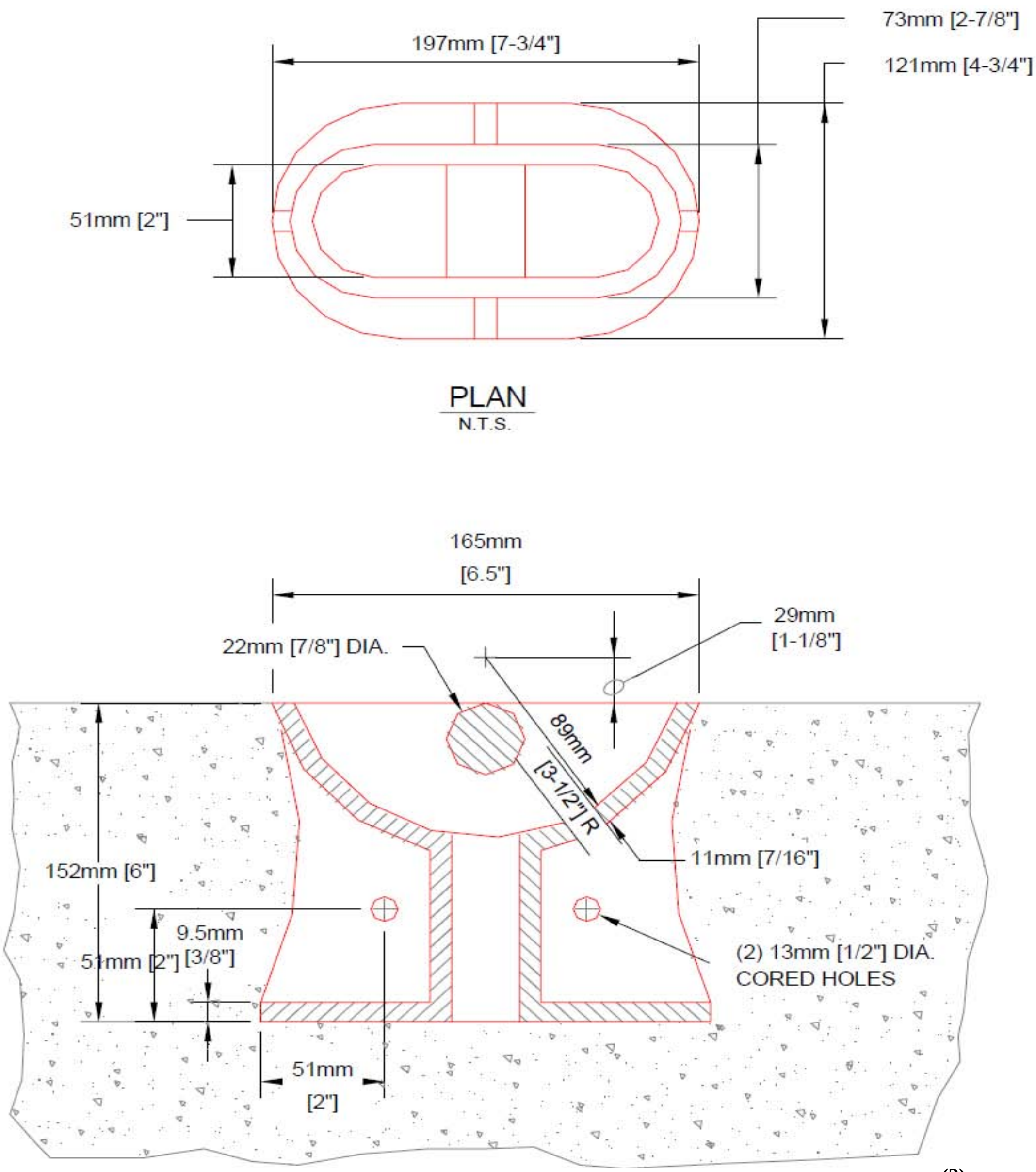


Figure 8. Plan and Elevation View of Installed Neenah Mooring Eye(2)

Twelve Neenah mooring points were installed on the AOS test pad. Four of the anchors had no pier, four of the anchors had a 4-ft-long pier, and the additional four anchors had an 8-ft-long pier. The objective was to determine the influence of various pier dimensions on the ultimate load capacity of each anchor. Each pier was circular and consisted of reinforced concrete. The reinforcement cage was constructed from ½-in-diameter reinforcement bar with each hoop spaced vertically at approximately one ft (Fig. 9). The figure below illustrates the rebar cage construction and placement. Each installation method is discussed in detail within this report.

Figure 9. Four-ft and 8-ft Rebar Cage with Sonotube Sleeve

5.1.1. Neenah Tie-downs Without Concrete Pier

Four Neenah mooring points were installed without a concrete pier. The installation process is discussed in the following sections.

Saw Cutting and Debris Removal. The initial procedure entailed removing a section of existing concrete that had been chosen as an anchor location; the term *existing concrete* refers to sections of the test pad that were not removed during the anchor installation process.

The section removal was accomplished with the use of a walk-behind diesel powered Husqvarna FS6600D concrete saw. This particular model had a power rating of 66 hp and powered a 42-in-diameter concrete cutting blade with maximum depth of cut of 17½ -in. After selection of the anchor location, a 3-ft by 3-ft square template was used to mark the perimeter of the concrete section to be removed. The saw operator ensured the saw blade was accurately aligned during the cutting process to eliminate binding the blade.

Also, to eliminate binding the saw blade, the saw-cutting procedure was performed with series of passes, with each subsequent pass increasing the overall depth of cut by approximately one third of the overall slab thickness. The first pass created a 4-in penetration and the second and third passes increased the depth of cut by approximately 4 in each.

It was important that the saw operator cut through the full slab depth before ceasing operations. It was difficult to determine the exact blade depth requirement on the final pass to ensure that the saw had achieved complete penetration into the upper layer of base course. However, the water discharge from the walk-behind saw changed colors after it penetrated the base course, and the saw blade resistance also decreased.

A skid steer with a blunt impactor pulverized the cut-out section into smaller segments to facilitate removal from the site. A 90-lb jackhammer connected to 110-psi air compressor provided an alternative to the skid steer for the pulverization process.

Base Course Preparation. Base course preparation was conducted subsequent to section cut-out and debris removal. A Wacker–Packer® engine-driven plate compactor consolidated and densified the exposed crushed-aggregate base course. This was important to guarantee compaction of the base course and alleviate settlement and load transfer concerns.

Drilling Rebar Holes. The Neenah anchor installed without a PCC pier performed most effectively when it was integrated into the existing concrete structure. This was accomplished by using 3/8-in-diameter dowel rods to mechanically connect the Neenah anchor to the existing PCC slab. To install the dowel rods a Hilti drill was utilized. Two dowel sleeves were drilled in two opposite sections of the slab (Fig. 10) for a total of four dowel sleeves per anchor.

Figure 10. Neenah Tie-down without Concrete Pier, Prior to PCC Placement

Neenah anchors are equipped with two ½-in-diameter cored holes to allow insertion of a section of rebar through the Neenah anchor to integrate the tie-down into the existing PCC. It was important that the rebar segment extended several in into the PCC slab. To facilitate this process, a Hilti drill was utilized to perform the concrete drilling process. The drill bit diameter exceeded the rebar diameter by a factor of three, allowing for the freshly placed PCC to completely fill the dowel sleeves. The drilled holes extended approximately 1 ft into one side of the slab and 6 in into the opposite side. One end of the rebar was inserted into the deeper hole, providing adequate space for the opposing end to be lowered into the excavated section and placed in the opposite, shallower hole. Before lowering the two pieces of rebar into the exca-vated section, the rebar was inserted through the appropriate cored holes on the Neenah anchor, in essence tying the mooring point into the existing PCC slab, as well as suspending the anchor.

Prior to exposing the Neenah mooring point to grout placement, the underside of the anchor was taped to eliminate grout intrusion into the hemispherical, open section. Additionally, the bowl of the tie-down was filled with a rag to eliminate concrete spilling into the mooring point. These precautions guaranteed adequate space for the cross-rod to physically attach to the tie-down connection elements, such as lifting shackles.

Grout Placement. After the anchor was set into place and the bowl section of the Neenah mooring point sealed, the excavated section was filled with freshly mixed concrete. A high-early-strength concrete mixture, with Type III cement, was used to expedite the installation and testing process. Type III concrete achieved its full specified strength in 4–7 days [6]. Conversely, it can be assumed that alternative mixes would not have achieved full specified strength until 28 days after placement[7]. This could be problematic in instances where the mooring point was likely to be loaded within a few days after installation. Alternative, rapid-setting grouts were available, though experience has shown their performance to be somewhat variable.

The concrete mix specified was a typical ¾-in-minus aggregate mix. Because limited space existed in the holes drilled into the slab it was imperative that coarse aggregate particles not become lodged against the rebar sections inside the dowel sleeves, impeding the flow of concrete and possibly creating air voids.

To facilitate rapid testing of the Neenah anchors a 5000-psi, Type III high-early-strength-concrete mix was specified. Alternatives considered included a higher strength (6000 psi, 7000 psi, etc) type I or II mix. Table 3 illustrates various strength gain ratios and the time required for different mixes to achieve the requisite compressive strength of 5000 psi. Figure 11 exhibits a Neenah Mooring Eye after PCC placement.

Table 3. PCC Strength Gain Ration

Type	Strength Ratio				Cure Time at Specified f'_c[28] (days)		
	3 day	7 day	14 day	28 day	5000 psi	6000 psi	7000 psi
I, II	N/A	0.67	0.86	1	28	12–14	7
III	0.88	1	1	1	7	3	2

Figure 11. Neenah Mooring Eye, Post-PCC Placement

5.1.2. Neenah Tie-downs with Concrete Pier

Eight Neenah mooring points were installed with concrete piers. Four of the anchors had piers that extended 4 ft below the pavement surface, and the remaining four piers extended 8 ft below the pavement surface. The installation procedures were basically identical for each pier length, thus only installation of the 4-ft pier is described in detail within this report.

Installation Procedures. The initial installation procedure for Neenah anchors with concrete piers was similar to the installation without piers. A section of concrete was cut away and removed. However, the process changed significantly subsequent to section removal.

The first major variation related to integration of the Neenah mooring point into the existing concrete matrix. Reinforcement bar was eliminated, significantly altering the load transfer mechanism between the anchor section and the existing PCC. The objective was to streamline and expedite the installation process.

After excavation of the anchor location, a 2-ft-diameter pier cavity was augered to accommodate insertion of the concrete pier reinforcement cage. Pier depths measured 4 ft and 8 ft. A line truck equipped with a 24-in-diameter auger performed the augering operations. Installation time was only slightly impacted by increasing the depth of the shaft from 4 to 8 ft. Figure 12 illustrates the augering process.

Figure 12. Augering Hole and Inserting Rebar Cage for PCC Pier

The Neenah anchor with pier was set in place in a similar manner to the Neenah with no concrete pier. An improvised frame allowed for the mooring point to be suspended at the proper location. Current field installations often involve wet-setting the Neenah tie-down, which is the practice of placing the anchor in freshly placed concrete.

Each pier was equipped with a steel reinforcement cage. The reinforcement increases the tensile strength of the concrete matrix. The cage also added mass to the pier, an important consideration considering the additional mass likely increased the pull-out resistance of the anchoring system. Rebar cages were constructed with ½-in-diameter (#4) rebar. Mat grids were separated by approximately 1 ft, and the cage was held together with rebar ties.

The soil comprising the subgrade could be best characterized as poorly graded silty sand. Additionally, the water table varied seasonally and at times was within 2 to 3 ft of the surface. To mitigate shaft instability and water intrusion, a Sonotube sleeve was inserted into the cavity prior to insertion of the reinforcement cage and subsequent grout placement. Additional water intrusion into the freshly placed grout could possibly have diminished the ultimate compressive strength and stability of the concrete pier.

Grout Placement. A high-early-strength PCC mix was utilized to form the pier and the excavated anchor section. The mix design incorporated ¾-in-minus coarse aggregate particles. However, the coarse aggregate size was of limited concern as there were no dowel sleeves present in the existing slab. Therefore, alternative larger aggregate mixes were acceptable with this installation method.

5.2. Hat-Type Tie-downs

Hat-Type anchors are very similar to an anchor currently utilized in contingency environments by the Air Force, as described in UFC 3-260-01[(2)]. The mooring point is essentially a segment of solid round stock with the middle bent into the shape of a hat. The AFRL machine shop fabricated each hat-type mooring point used in the testing process. Figures 13 and 14 illustrate the physical dimensions of the tie-downs installed and load tested by AFRL on the AOS test pad. Overall height modifications may be required depending on the concrete depth. This anchor is not commercially available.

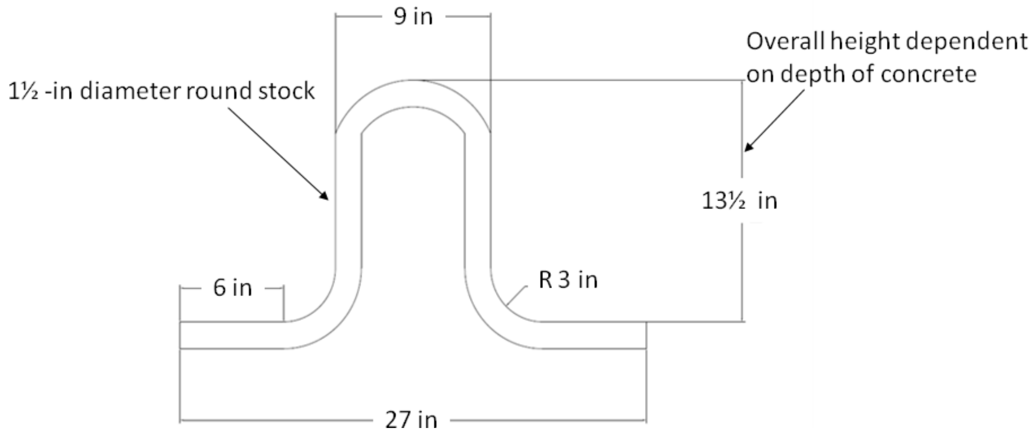


Figure 13. Hat-Type Tie-Down Schematic

Figure 14. Hat-Type Tie-down

Hat-type mooring points were installed using two alternate methods, each method using slightly different equipment. A detailed description of each method has been supplied with this report.

5.2.1. Hat-Type Tie-down Installation: Method A- Saw-cutting the Slab

Cutting and Debris Removal. This hat type installation method was similar to the Neenah anchor installation in terms of utilizing a walk-behind concrete saw to remove a section of the concrete slab for anchor placement. The initial process entailed identifying and marking an section sized adequately for removal. For the hat-type anchors tested, a section cut-out 1 ft wide was required. Requisite length was determined by measuring the mooring point and adding an additional ten percent to the cut-out length (Example: hat-tie length = 30 in, cut-out section length = 33 in). Considerations detailing the saw-cutting procedures and debris removal have been discussed in depth in a previous section of this report.

After cutting and removal of debris the next phase of installation entailed excavating 4–6 in of the base course material. The compacted base course material was extremely difficult and labor intensive to excavate. Ultimately, the excavation was accomplished with the use of a water distribution system and a high-powered water pump. In essence, a high-pressure fluid stream water-jetted the base course. It was important to excavate the exposed section of base course and to extend the excavation several in under the adjacent portions of the concrete slab. This allowed inserting the hat-type anchor and rotating it 90° in the removed section to locate the ends of the mooring point under the existing concrete slab. Prior to anchor placement, it was necessary to use a shop vacuum and a small bucket to remove water and debris from water-jetting.

The depth of excavation did not need to be precise, just adequate to allow for the hat-type anchor to be rotated under the slab. An important consideration that factored into the desired base course excavation was the thickness of the base course layer itself. It was not desirable to completely penetrate the base course layer and reach the sub-grade soil layer. The subgrade was not adequately compacted to carry the concrete layer, and could lead to voids under the slab. Figure 15 depicts the saw-cut method, hat-type anchor installation process.

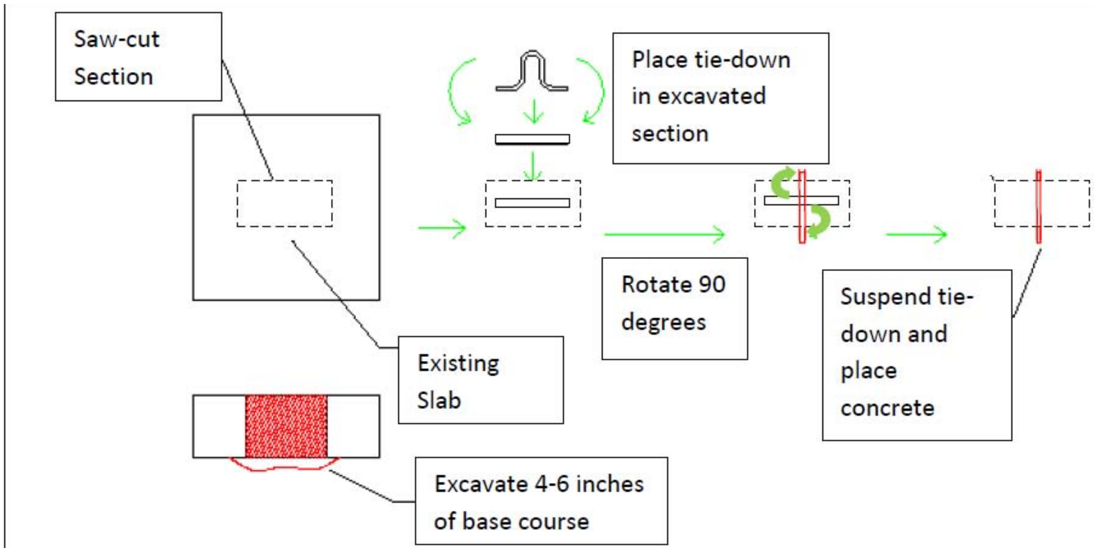


Figure 15. Hat-Type Anchor Installation Schematic: Saw-cut Method

Anchor installation procedures involved suspending the anchor at the proper height within the excavated section. To accomplish this, an improvised frame was arranged utilizing a standard segment of lumber and a tie wire to attach the mooring point to the improvised wooden frame. The top of the hat-type anchor was located approximately ¼ in below the top of the slab surface.

Grout Placement. After the mooring point was adequately secured to the frame high-early-strength PCC was placed into the excavated section. It was imperative to ensure the grout reached all the recesses present under the slab. For this reason, a ¾-in-minus mix design was specified, and a concrete vibrator was utilized to reduce air voids and encourage flow into the recessed areas under the existing concrete matrix.

Before the grout hardened it was necessary to form a recess around the top of the hat-type mooring point. This was accomplished by hand scooping a roughly 4-in-diameter hemispherical void (Fig. 16) in the freshly placed concrete. The void area allowed for adequate space to attach connecting members to the mooring point.

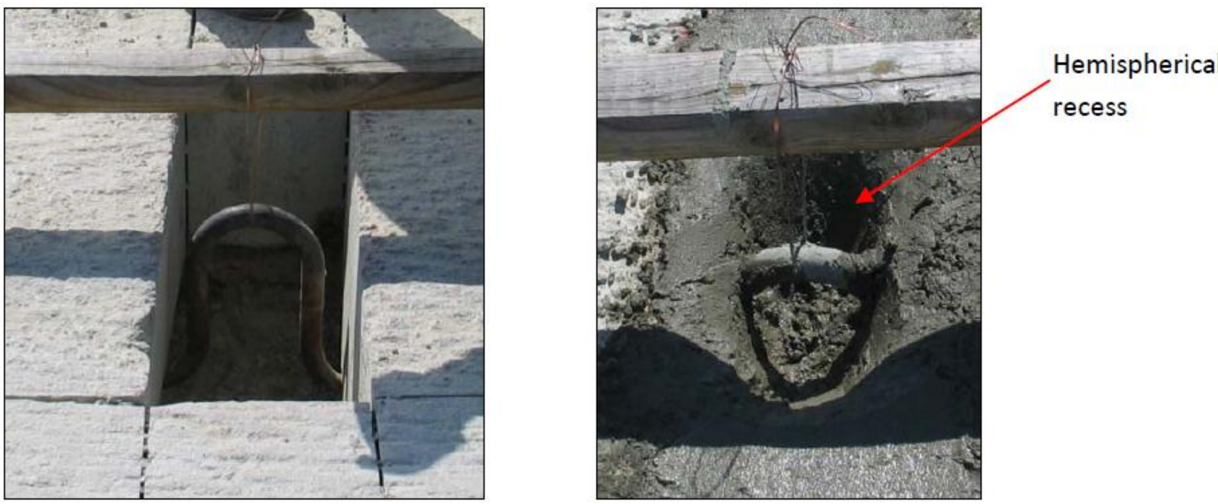


Figure 16. Hat Type Anchor- Pre and Post PCC Placement

5.2.2. Hat-Type Tie-down Installation: Method B- Coring the Slab

Coring and Core Extraction. This installation method mirrored Method A in most respects. The major delineation between the two methods involved slab removal. As opposed to saw-cutting, Method B incorporated a coring rig capable of cutting 1-ft-diameter core samples. Core samples were taken in such a manner that they overlapped each other (Fig. 17). Adjoining cores provided sufficient length and width to insert the hat-type mooring point. Following extraction of three adjacent cores, the installation procedure (Fig. 18) was identical to those described in Method A.

Method B allowed for a reduction in equipment. The concrete saw and Bobcat were not necessary (90-lb jackhammer and 110-psi air compressor also not necessary if taking the place of Bobcat attachments). However, a coring rig capable of accepting 1-ft-diameter coring bits was necessary. Some facilities may not have access to this type of coring rig.

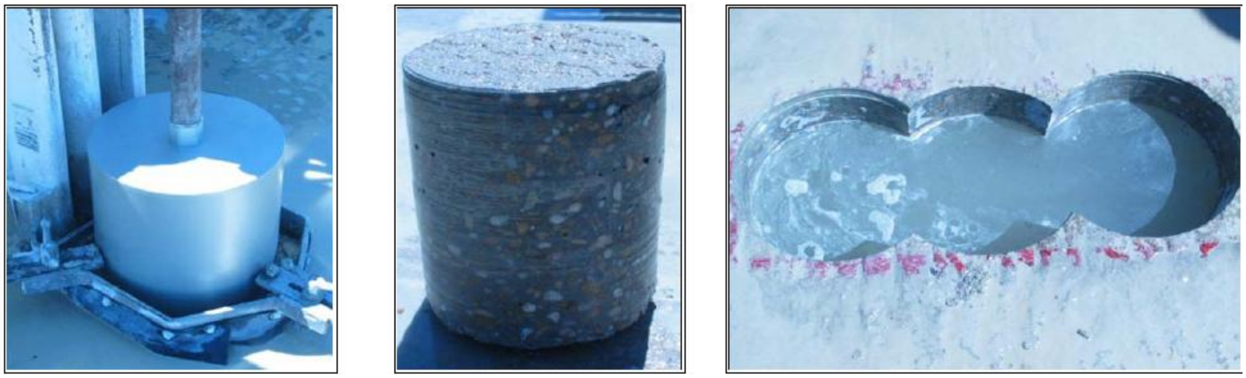


Figure 17. Coring Operation and Extraction

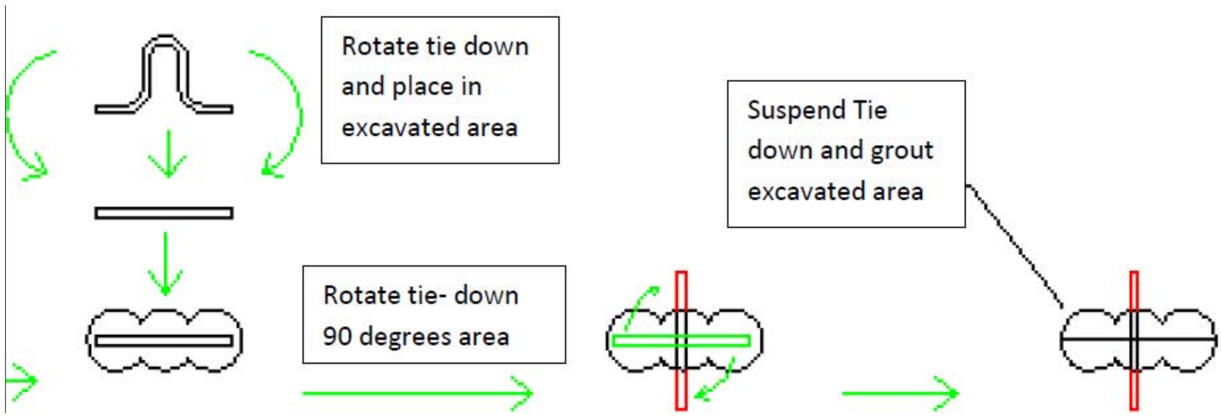


Figure 18. Hat Type Anchor Installation Schematic- Coring Method

5.3. Contingency Concrete Anchors: Results and Discussion

Table 4 provides uplift capacities for the various installation configurations incorporating Neenah mooring eyes.

Table 4. Pull-out Capacity of Neenah Mooring Eye Test Samples

Length of Pier under Neenah Anchor:	Load at Failure (lbs)				
	Test 1	Test 2	Test 3	Test 4	Mean
None	37,000	37,000	37,000	37,000	37,000
4 ft	37,000	37,000	37,000	37,000	37,000
8 ft	46,728	50,000	46,079	45,430	47,214

5.3.1. Neenah Anchors without Pier

Four Neenah tie-downs without a reinforced concrete pier were installed and subsequently load tested. The installation procedure has been detailed in a previous section of this report. Each of the mooring points failed at a load of approximately 37,000 lbs.

The observed failure mode for each anchor was fracture of the cast iron cross rod. No upheaval of the surrounding surface was evidenced. It is important to note that Neenah Foundry, the manufacturer, specifies a 9,000-lb capacity for the cross rod. After testing, it is assumed that this value takes into account a 4:1 to 5:1 safety factor.

5.3.2. Neenah Anchors with 4-ft Reinforced Concrete Pier

A total of four Neenah aircraft mooring eyes were installed in conjunction with a 4-ft reinforced concrete pier and subsequently load tested. These anchors did not incorporate rebar segments to tie into the existing concrete test pad. Interestingly, the pull-out capacity afforded by this particular tie-down configuration was 37,000 lbs, identical to the pull-out capacity of the Neenah anchors installed without an accompanying pier. The observed failure mechanism was also identical, a fracture of the cast iron cross-rod.

5.3.3. Neenah Anchors with 8-ft Reinforced Concrete Pier

A total of four Neenah aircraft mooring eyes were installed in conjunction with an 8-ft reinforced concrete pier and subsequently load tested. Like the mooring points with a 4-ft pier, this arrangement did not utilize rebar segments to tie into the existing concrete matrix. Testing results proved to be perplexing. The observed failure mechanism was the same as the other two groups of Neenah anchors (Fig. 19). However, the average pull-out capacity of this installation design measured 47,214 lbs—a significant increase over the other two groups. Additionally, the data do not appear to be skewed by outliers. The measured vertical resistances measured from 45,430 lbs to 50,000 lbs.

Figure 19. Neenah Cross-Rod Failure

5.3.4. Saw-cut Hat-Type Tie-downs

One saw-cut hat type anchor was installed on the AOS test pad. The installation procedure has been discussed in detail in a previous section of this report. After installation, the anchor was subjected to a vertical tensile force in an attempt to determine the pull-out capacity of the mooring point. The anchor did not exhibit any deflection, and the surrounding slab did not appear to up-heave. Testing was terminated at 50,000 lbs, the limit of the testing equipment.

5.3.5. Cored Hat-Type Tie-downs

Three cored hat-type mooring points were installed on the AOS test pad and load tested. Installation procedures for this specific anchor set-up have been provided within this report. Each of the anchors was loaded to the extent of the testing equipment (50,000 lbs). Encouragingly, there was no upheaval in the surrounding surface and none of the mooring points measurably yielded.

5.4. Contingency Concrete Anchors: Conclusions and Recommendations

5.4.1. Heavyweight Tie-downs

Hat-type mooring points represent the best option for heavyweight contingency concrete anchors for use in jointed plain concrete sections at least 12 in thick. The tie-down capacity exceeded 50,000 lbs, and the installation times were shorter than for Neenah mooring points (with an 8-ft concrete pier). Additionally, there was no variance in the capacity afforded by either of the two hat-type installation methods. Individual facilities are likely to have the requisite equipment to perform one of, if not both, of the installation procedures.

5.4.2. Lightweight Tie-downs

In PCC sections at least 12 in thick, Neenah anchors installed without a concrete pier represent the optimal lightweight contingency tie-down. This mooring point is relatively simple to install and provides a pull-out capacity that far exceeds the lightweight criteria but does not meet the heavyweight criteria.

Additional testing is necessary to determine the impact of eliminating the rebar segment. Until such further testing, the rebar installation should not be eliminated to expedite the install process. Also, it is recommended that further testing be conducted with Neenah anchors set in place utilizing the coring method to determine the pull-out capacity and time required for this particular set-up.

In PCC sections less than 12 in but more than 6 in thick, Neenah tie-downs installed with 8-ft piers represent the best option. Four tests were conducted on Neenah mooring points installed with an 8-ft reinforced concrete pier. The expected pull-out capacity was 47,000 lbs. This value met the pull-out capacity required for heavyweight mooring points. However, due to the failure mechanism of the mooring eye, a conservative approach is recommended. Therefore, this mooring point is not suggested to serve in a heavyweight loading role, but rather in a lightweight mooring function, serving in rigid pavements less than 12 in but more than 6 in thick.

6. PHASE THREE: CONTINGENCY ASPHALT ANCHORS

The design process for testing tie-downs in contingency asphalt environments encompassed several themes. The ultimate objective was to develop and test anchoring systems capable of meeting the lightweight and heavyweight anchoring requirements specified by AFCESA. Lightweight anchors are defined as tie-downs that provide an ultimate uplift resistance of 17,000 lbs, while heavyweight anchors provide an ultimate uplift resistance of 37,700 lbs.

The current Air Force method for installing asphalt tie-downs is to excavate a section of the sub-grade material and place fresh concrete in the resulting void. The dimensions of the excavated section provide volume adequate that the weight of the concrete pier itself is capable of providing the necessary pull-out resistance(2). For example, a 6-ft by 6-ft section excavated to a depth of 7 ft and subsequently filled with PCC provides enough weight to resist a pull-out force of 37,800 lbs.

The focus of this testing was to develop less labor-intensive anchoring systems and installation methods that use readily available equipment and materials. Readily available equipment is best defined as equipment that military facilities have reasonable access to regardless of geographical location, such as a skid steer and basic field implements. In contrast, individual facilities may not have access to equipment with deep augering capabilities, a coring rig, or other various specialized equipment. This phase of the project focused on developing anchoring systems for widespread use amongst the various military branches, irrespective of location.

Several commercially available mooring points were tested, in addition to tie-downs developed within the military service and research branches. The installation process focused on mooring points that required equipment and equipment capabilities that individual military installations would likely have access to. Additionally, tie-down testing was conducted at several locations, each with a distinct soil profile. This was important because the asphalt matrix provides little resistance and the soil matrix is responsible for a vast majority of the load resistance. Load testing was performed at 1) Silver Flag Exercise Site, Tyndall AFB, FL; 2) Seguin Auxiliary Airfield, Seguin, TX; and 3) Avon Park Air Force Range, Avon Park, FL.

6.1. Flexible Pavement Tie-downs

Several asphalt anchoring systems were chosen for load capacity testing. The types of anchors chosen varied in design complexity, installation depth, and installation method. Anchors were installed and load tested in three distinct soil profiles, varying from silty clay (Seguin Auxiliary Airfield) to poorly graded silty sand (Silver Flag Exercise Site). Ultimate uplift capacity and deflection data were collected at each location.

Manta-Ray SR (MR-SR) earth anchors, fully grouted piers, partially grouted piers, U.S. Army Engineering Research and Development Center (ERDC) tri-talon anchors, and AFRL epoxied anchors were chosen for this evaluation. Several additional tie-downs were deselected from further consideration due to concerns regarding equipment availability in contingency environments. The majority of the selected flexible pavement mooring points relied on soil conditions rather than asphalt conditions. The exceptions were the epoxied anchor systems and the tri-talon anchors, which, due to their shallow installation depth, were both heavily reliant on the base course and the asphalt matrix.

Each tie-down and its respective installation procedure is discussed in detail, including anchor performance. Most soil anchors can be constructed and installed in a variety of ways. It was important to minimize the amount of specialized equipment and materials used to build and install the tie-downs. Therefore, installation techniques requiring a minimal logistical footprint were a necessity. Installation timelines and equipment lists for flexible mooring points have been provided in Appendix C of this report.

6.1.1. Fully Grouted Piers

A grouted anchor is an anchor in which a metal tendon is inserted into a drilled hole and then grouted[8]. These anchors have a wide range of commercial uses, and the size and complexity of the anchor(s) vary considerably with application. Figure 20 shows several different types of grouted anchors.

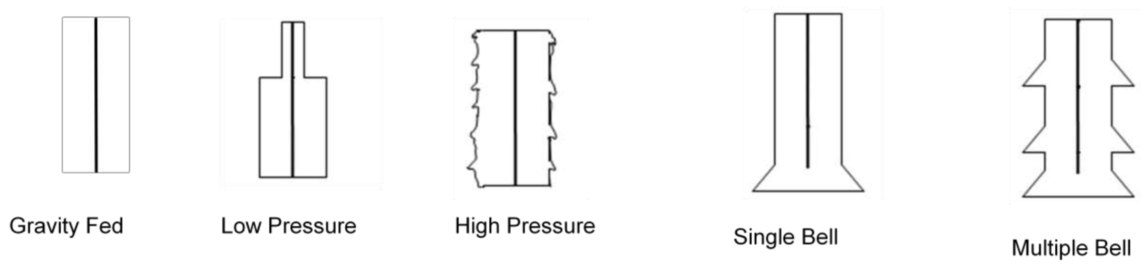


Figure 20. Grouted Anchor Installation Methods

Fully grouted pier tie-downs are set by excavating the existing soil and placing fresh concrete in the resulting void. Typically, a hole is augered or drilled into the base material where the anchor is to be placed. A metal tendon is then centered in the hole, followed by grout placement.

Uplift resistance of the pier is dependent on several factors; installation technique, pier geometry, drilling and grouting method, grouting pressure, and engineering properties of the in-situ soil[9]. Freshly placed concrete exerts pressure (the amount of pressure depends on the injection method) and interacts with the adjacent soil matrix, bonding with the soil to various degrees. Steel reinforcement, if present, adds additional mass to the column and increases the uplift capacity.

Pull-out capacity of grouted piers is difficult to predict. There are numerous soil parameters that impact the ultimate uplift resistance; these parameters can be complex and hard to quantify. Predictive models exist for determining pile capacity, though these models are designed for much larger, load-bearing piers. One relatively simple pile model was investigated to simulate the pull-out capacity of fully grouted piers in various soil conditions. The model was a composite based on literature presented by two different authors, and incorporated pier weight and skin friction developed along the pier/soil interface. Ultimate pull-out capacity of a grouted pier is governed by the weight of the pier and the skin friction force developed along the length of the pier[10]. The skin friction force is dependent on several factors including soil type, soil engineering properties, pier diameter, length of pier embedment, and the location of the water table[11]. Empirical skin friction values, *s*, compiled from field testing, are provided in Table 5[12].

The soil classifications in the chart were derived from *N* values determined by standard penetration test (SPT) blow counts. Predictions are compared to measured pull-out capacities in Appendix D of this report. The skin friction can be calculated for a pier by selecting the soil type in Table 5 and multiplying it by the surface area of the pier[13].

Table 5. Empirical Skin Friction Values for Various Soil Types

Soil Condition	Ordinary Range of *s*, lb/ft^2
Cohesive Soils	
Silt	300 ± 200
Soft Clay	400 ± 200
Silty Clay	600 ± 200
Sandy Clay	600 ± 200
Medium Clay	700 ± 200
Sandy Silt	800 ± 200
Firm Clay	900 ± 200
Dense Silty Clay	1200 ± 300
Hard (Stiff) Clay	1500 ± 400
Cohesionless Soils	
Silty Sand	800 ± 200
Sand	1200 ± 500
Sand and Gravel	2000 ± 1000
Gravel	2500 ± 1000

Current Air Force guidelines for flexible pavement tie-downs consist of a concrete column of sufficient mass to provide the requisite uplift resistance without depending on soil friction, or any other factors. Pier geometry is dependent on the pull-out demand. To meet the heavyweight loading criteria, UFC 3-260-01 specifies a 6-ft by 6-ft by 7-ft pier. Considering that PCC weighs approximately 150 lb/ft^3, the specified column weight is 37,800 lbs. Decreased load demands can be met by constructing a pier with less volume.

The primary objective of AFRL fully grouted pier anchor testing was to develop a less labor intensive installation method than the current pier installation procedure—one that can be performed in various soil conditions, while still meeting the requisite loading requirements. It was also important to cultivate an expeditious installation routine that relies on equipment likely to be readily available at most facilities.

The selected column consisted of an 8-in-diameter round pier, 6–7 ft deep. The decisive factor determining the pier geometry was equipment availability in contingency environments, specifically skid steers. It is likely that most airfield environments will have access to a skid steer, such as a Bobcat. Skid steers are limited in capacity and these limitations factored into the design considerations.

Piers are formed by excavating the existing soil. One of the simplest excavation procedures utilizes an augering system to create a pier cavity. After a sufficient cavity has been augered, the void is completely filled with PCC. The result is a concrete pier. The depth of the pier is limited by the height restrictions of the augering system. Depending on the skid steer, it is difficult to auger a cavity more than 6–7 ft deep. Additionally, individual contingency facilities are likely to have access to an 8-in-diameter auger. Larger-diameter augers may be difficult to appropriate, particularly in contingency environments. These factors led to the development of the AFRL fully grouted pier.

6.1.2. Manta Ray Earth Anchors (MR-SR)

MR-SR earth anchors are driven plate soil anchors. They can also be characterized as direct embedment anchors. The concept of the direct embedment tie-down is to drive the anchor into the soil until the anchor reaches the intended depth. Upon achieving the requisite depth the plate is pulled up and locked into position. This in effect creates a plate anchor with a large subgrade failure plane. Figure 21 illustrates the basic installation concept.

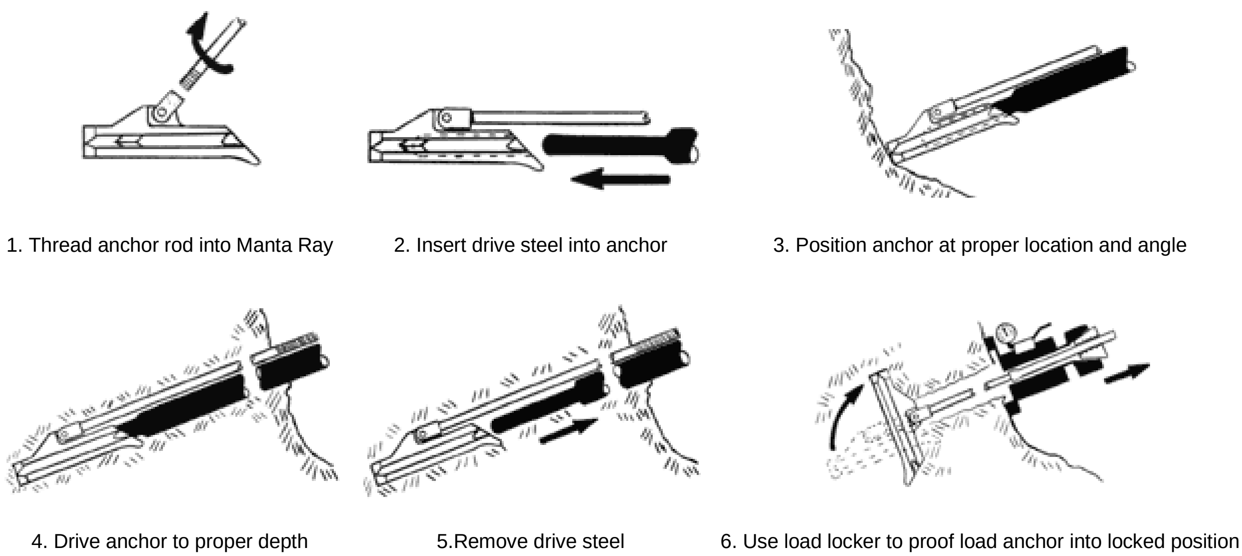


Figure 21. Manta Ray Installation Process[14]

Several methods are available for predicting the uplift capacity of direct-embedment anchor systems in sand. It is important to note that these predictive models are based on predicting plate anchor strength. Each method ultimately models the capacity of the plate anchor by attempting to determine the failure plane of an anchor from its size, shape and the soil conditions of its environment. Meyerhof and Adams' method is the lone method that addresses rectangular anchors, as opposed to the other models which only address circular anchors. Das and Meyerhof have each developed predictive models for determining uplift capacity of plate anchors in cohesive soils[14]. Predicting the uplift capacity of MS-RS tie-downs is difficult due to the geometry of the anchor.

Manta Ray mooring points are driven with conventional hydraulic or pneumatic equipment that is likely to be readily available in contingency environments. The Manta Ray line includes several different tie-downs, varying by surface area. According to the manufacturer, depending on anchor type and soil conditions, certain Manta Rays are capable of an ultimate uplift capacity of 40,000 lbs[14]. The manufacturer of Manta ray tie-downs also produces a higher-capacity mooring point, Stingray anchors. Stingray anchors are similar to Manta Ray anchors, except Stingray tie-downs have a much larger surface area.

The same concerns regarding equipment availability factored into the selection process when considering the specific Manta Ray or Stingray anchor to conduct testing. The manufacturer of both anchoring systems expressed concerns about the ability of the skid steer to hydraulically drive Stingray anchors, suggesting one of the larger Manta Ray mooring points. Individual facilities may have access to the requisite equipment necessary for the installation of Stingray tie-downs. However, this equipment is less likely to be readily available in contingency environments. Therefore, Stingray anchors were deselected from further consideration. The specific tie-down selected was the MR-SR.

The MR-SR tie-down is the largest Manta Ray anchor. The ultimate capacity of the MR-SR mooring point, according to the producer, is 40,000 lbs (Table 6). This value is dependent on soil conditions and depth of installation. Typical installation depths range from 7 to 30 ft (Fig. 22), although deeper installations are possible. Installation depths are typically determined pre-anchor placement and are based on comprehensive soil boring logs and SPT blow count data. Contingency environments likely will not have access to this information. Therefore, an arbitrary installation depth of 12 ft, within the typical range, was chosen for each location.

Table 6. Theoretical Manta Ray Pull-out Capacities in Various Soil Conditions

Soil Description	Blow Count	Stingray			Manta Ray			
		SR-3	SR-2	SR-1	MR -SR	MR-1	MR-2	MR-3
		Pullout Capacity (kips)			Pullout Capacity (kips)			
Dense fine compact sands, very hard silts or clays	45–60	100[2),3)]	79–89[2),4)]	58–65[2),4)]	40[1),3)]	36–40[1),3),4)]	21–28[2),4)]	17–20[2),3),4)]
Dense clays, sands and gravels, hard silts and clays	35–50	85–100[2),3),4)]	62–79[4)]	39–58[4)]	32–40[2),3),4)]	24–36[2),4)]	15–22[2),4)]	12–18[2),4)]
Medium dense sandy gravel, stiff to hard silts and clays	24–40	63–90[4)]	46–66[4)]	29–41[4)]	24–34[2),4)]	18–20[2),4)]	12–18[4)]	9–14[4)]
Medium dense coarse sand and gravel, stiff to very stiff silts and clays	14–25	48–63[4)]	31–48[4)]	24–32[4)]	18–24[4)]	15–20[4)]	9–12[4)]	7–9[4)]
Loose to medium dense fine to coarse sand, firm to stiff clays and silts	7–14	37–48[4)]	27–36[4)]	16–24[4)]	14–18[4)]	10–15[4)]	7–10[4)]	5–8[4)]

Notes:

1) Drilled pilot hole required for efficient installation.
2) Ease of installation may be improved by drilling a pilot hole.
3) Holding capacity limited by ultimate strength of anchors.
4) Holding capacity limited by soil failure.

6.1.3. Partially Grouted Piers

Partially grouted piers are similar to fully grouted piers. The major difference is that the excavated cavity is only partially filled with grout. The remainder of the cavity is backfilled with material removed during the excavation process. This soil is then compacted, though likely not to the original state. The dimensions of the cavity for the partially grouted pier are identical to the dimensions of the fully grouted pier cavity. The same design considerations factored into the selection of this particular anchor type, logistical footprint and ease of install. These anchors have been used successfully in environments where access to specialized equipment and materials is non-existent [15].

The uplift capacity of partially grouted anchors is difficult to predict. It is possible to model the pull-out capacity if partially grouted tie-downs are considered as circular plate anchors. Traditional, and more recent models, attempt to predict the capacity of plate anchors. Early theories, including the Soil Cone Method and Friction Cylinder Method, were relegated to shallow circular anchor plates, as opposed to more contemporary models which take square

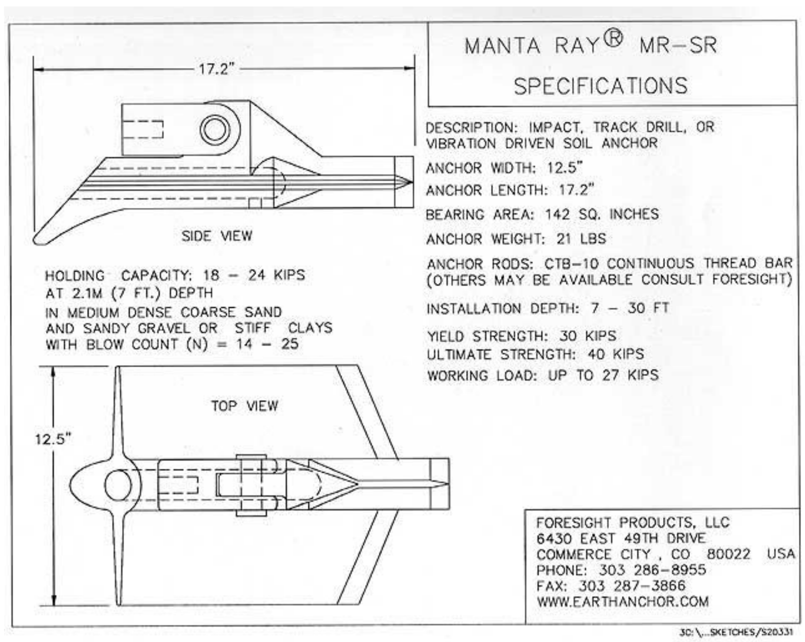


Figure 22. Manta Ray MR-SR Technical Specifications

anchors into account as well. The Soil Cone Method and Friction Cylinder Method each modeled a different soil failure surface (Fig. 23). The uplift resistance is assumed to be equal to the weight of the soil located inside the failure surface, as well as the frictional resistance developed along the failure plane[16].

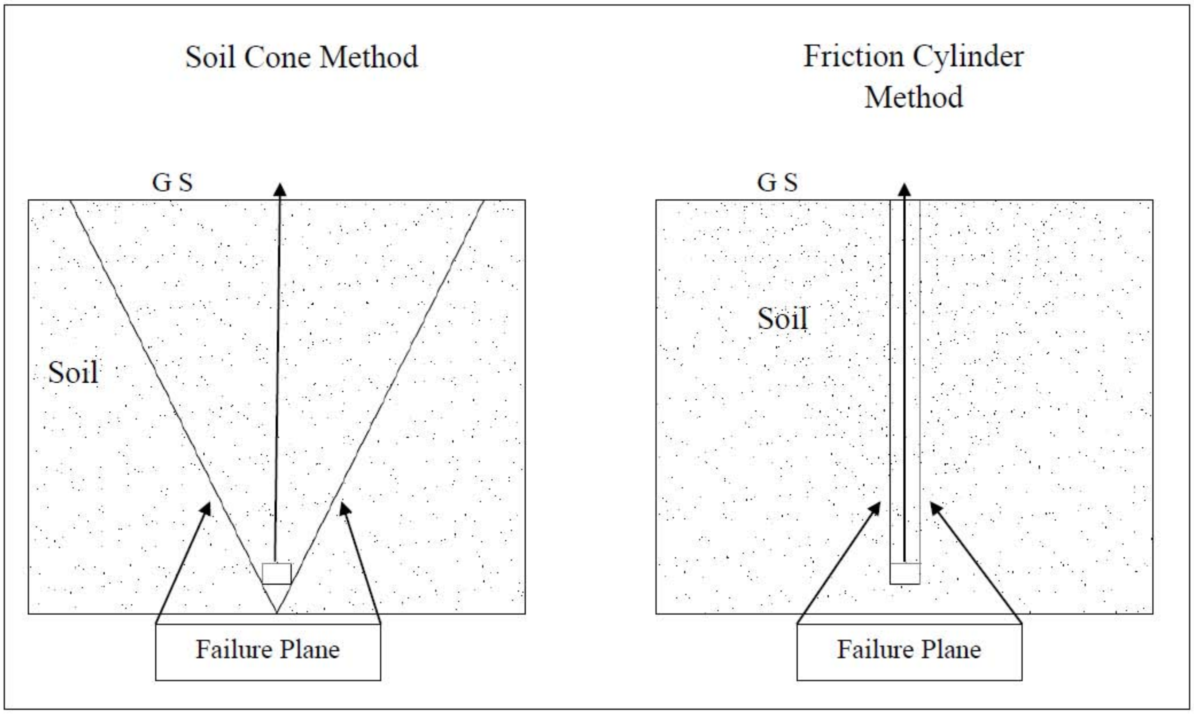


Figure 23. Assumed Soil Failure Planes for Soil Cone and Friction Cylinder Method[16]

The Soil Cone Method assumes a soil failure plane that can be approximated by a truncated cone, while the Friction Cylinder Method's failure plane is represented by a soil cylinder. More recently, several semi-theoretical/semi-empirical models have been developed to predict the pull-out capacity of shallow anchor plates in cohesive and cohesionless soils. However, models for cohesionless soils are much more prevalent. The various models are all based on the following parameters: the size of the excavated shaft and the soil strength parameters of both the host and backfilled soils[17].

6.1.4. Tri-Talon Anchors

Tri-Talon anchors were developed by ERDC. The original intent of these tie-downs was to secure folded fiberglass matting (FFM) to airfield pavement surfaces. FFM is utilized as a temporary repair surface for damaged airfield sections. Tri-Talon anchors were designed as a simple and expeditious method of securing the FFM to the sub-grade material.

ERDC conducted several vertical pull tests on the Tri-Talon anchors. The results were encouraging—individual anchors achieved pull-out capacities in excess of 17,000 lbs, exceeding the lightweight anchoring criterion [18]. Deflection data were not available. Figure 24 illustrates a Tri-Talon anchor.

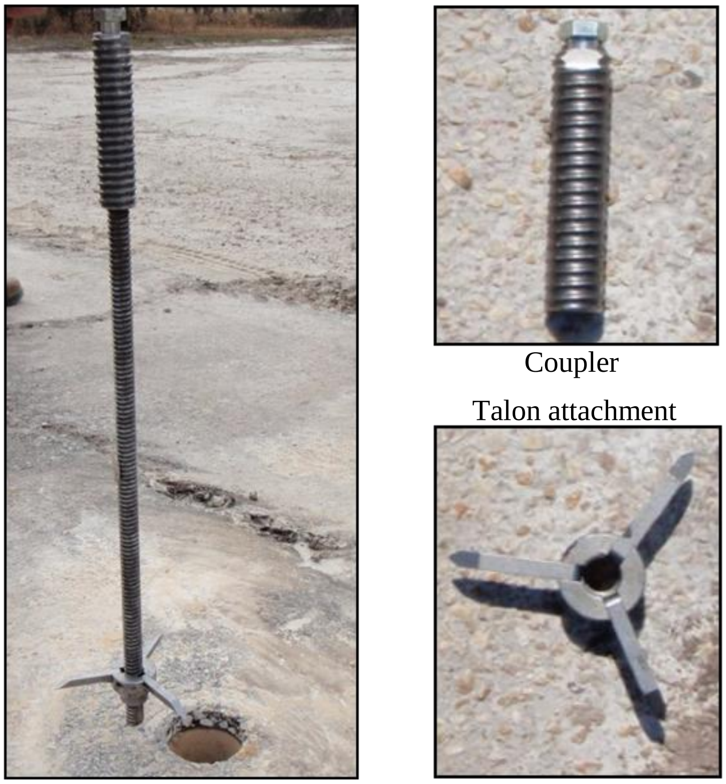


Figure 24. Tri-Talon Tie-down

6.1.5. AFRL Epoxied Anchors

AFRL epoxied anchors were developed by AFRL, also for use with FFM, and are in effect a modified Hilti HSL M12/50 expansive concrete wedge anchor. The anchors are approximately 9.5 in (241 mm) long and 0.5 in (13 mm) in diameter. The outside diameter of the sleeve is 0.75 in (19-mm). Figure 25 shows the standard anchor and its components: a shoulder bolt, washer, steel sleeve, nylon collar section, expansion sleeve, and cone. The modified anchor is created by removing the nylon collar section, expansion sleeve and cone. After removing the aforementioned components, three ½-in washers (inside diameter) are placed on the threaded portion of the expansion bolt. Each washer is separated by a 5/8-in coupling nut. The coupling nuts are oversized and not threaded, serving as a spacer between washers. The center washer is located approximately 8½ in from the bolt head. Two M12 nuts are threaded to the bottom portion of the anchor bolt to hold the components in place. Figure 26 illustrates the modified anchor. After the cavity was drilled, epoxy was inserted and the modified wedge bolt was set in place.

Figure 25. Traditional Hilti HSL M12/50 Wedge Anchor

Figure 26. Modified Hilti HSL M12/50 Wedge Anchor (AFRL Epoxy Anchor)

The design concept uses the washers to provide a larger bearing surface for the epoxy to adhere to. Installation requires drilling a 2-in-diameter hole 18 in through the pavement surface and the sub-grade material. The major design element is an undercut section in the sub-base or sub-grade material, at approximately the same depth as the center washer. This undercut creates a soil bulb for epoxy to infiltrate. The resulting epoxy bulb increases the failure plane and thus the load-carrying capacity of the tie-down. Load testing previously conducted on the modified Hilti HSL M12/50 anchors, installed in a 4¾-in flexible pavement surface underlain by a 9-in crushed-stone base showed pull-out capacities ranging from 5,000 to 15,000 lbs, with an average of 10,000 lbs[19].

6.1.6. AFRL Rapid Set Anchors

AFRL rapid set anchors are an exact replica of the AFRL epoxy anchor. The difference is that rapid set anchors are installed in a shaft filled with rapid-setting grout. The drilled cavity is identical to the one described in the previous section.

6.1.7. Modified AFRL Rapid Set Anchors

Modified AFRL rapid set anchors, shown in Figure 27, consist of a partially threaded 5/8-in anchor bolt. The threaded portion of the bolt contains two 5/8-in flat washers separated by a ¾-in nut. The components are held in a fixed position by a 5/8-in threaded nut. The anchor is placed in a 2-in-diameter, 18-in-deep drilled cavity that is then filled with grout. This anchoring system also incorporates an under-cut section. The objective is to determine if the larger diameter bolt measurably increases the pull-out capacity of the anchoring system.

Figure 27. Modified AFRL Anchor

6.1.8. AFRL Epoxy Anchor Plates

AFRL epoxy anchor plates are similar to the individual epoxy anchors, except that the plates are used to group multiple epoxy anchors together. Two different plate geometries were designed to be tested. One plate measures 12 in by 12 in, and the other plate measures 18 in by 18 in. Each plate is 1 in thick and has four holes, one near each edge, to mount bolts. Figure 28 illustrates plate dimensions and anchor spacing distances for each of the two plate sizes.

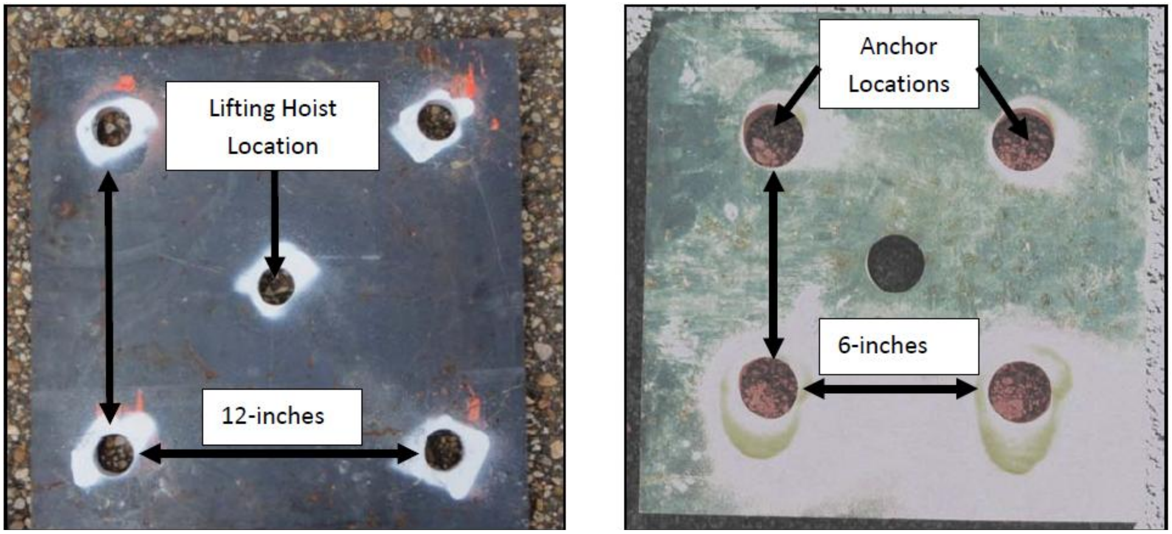


Figure 28. AFRL Epoxy Anchor Plates

Previously conducted individual AFRL epoxy anchor testing established a mean pull-out capacity of 10,000 lbs, below the lightweight anchor threshold. The objective of the epoxied anchor plate testing is to determine if a group of individual anchors working in concert with one another increases the ultimate pull-out capacity sufficiently to meet the heavyweight and/or lightweight loading criteria. Additionally, the different-sized plates allowed for two separate anchor spacing options. It was important to determine if anchor spacing had a significant impact on the efficiency of the anchor group. This again refers to the failure zone of each individual anchor.

The failure region is a theoretical cone shaped zone that extends from the anchor plate—in this instance the washer/soil bulb location—to the pavement surface. Theoretically, if the anchors are spaced far enough apart, the influence zones for each anchor will not converge and the group will approach 100 percent efficiency.

6.2. Testing Locations

Aircraft tie-down testing was conducted at three locations: 1) Silver Flag Exercise Site, Tyndall AFB, FL; 2) Seguin Auxiliary Airfield, Seguin, TX; 3) Avon Park Air Force Range, Avon Park, FL. Subgrade soil type and base course composition differed at each of these locations. However, the pavement condition was similar at each site. Asphalt surfaces were well worn, with thicknesses measuring 1–3 in. Installations were performed in airfield sections no longer exposed to aircraft traffic.

The objective of the flexible pavement tie-down phase of the project was to develop flexible pavement anchors that could be installed in any environment and still retain full functionality. For this reason sub-optimal pavement sections were desirable in that they represented the worst-case scenario. Additionally, it was imperative to measure the performance of competing soil anchoring systems installed in different soil types. Therefore, three testing locations were chosen due to their various soil profiles. The following sections discuss each location and the tie-down installation in depth.

6.2.1. Silver Flag Exercise Site

Tyndall Air Force Base, established in 1941, is located in the Florida Panhandle, near the Gulf of Mexico. Tyndall AFB is on a peninsula with a southeast/northwest orientation that separates the Gulf of Mexico from St. Andrews Bay to the northeast.

Climate in the Tyndall AFB area are characterized by warm, humid summer temperatures and cool to mild winter conditions. Average yearly precipitation is 53.2 in. The wet season extends from May through September, and during this period scattered afternoon thundershowers are a common occurrence. Hurricanes pose a threat to the base from June through November, with the greatest possibility of occurrence in September. Peak wind gusts of 69 knots have been measured at Tyndall AFB. The base elevation is 15 ft to 20 ft above sea level.

Silver Flag is a 1,200-acre exercise and training site situated in a partially cleared wooded area of the base. The flexible pavement surface is in poor condition and 1¾–3 in thick. The pavement surface is situated on a 6-in-thick crushed aggregate base course layer. The subgrade material is poorly graded silty sand, with a water table that varies from 2 ft to 5 ft below grade. The high water table, fairly incompetent soil profile, thin base course layer, and worn condition of the thin flexible pavement surface provide worst-case testing conditions. Anchor installation at the Silver Flag Exercise Site included the following tie-downs:

a. Fully grouted piers
b. Partially grouted piers
c. MR-SR soil anchors
d. AFRL epoxied anchors
e. AFRL epoxied anchor plates
f. AFRL rapid-set anchors
g. AFRL modified rapid-set anchors
h. Tri-talon anchors

Installation procedures for each anchor and load testing results are presented in detail in subsequent sections of this report. The Silver Flag test layout is shown in Figure 29.

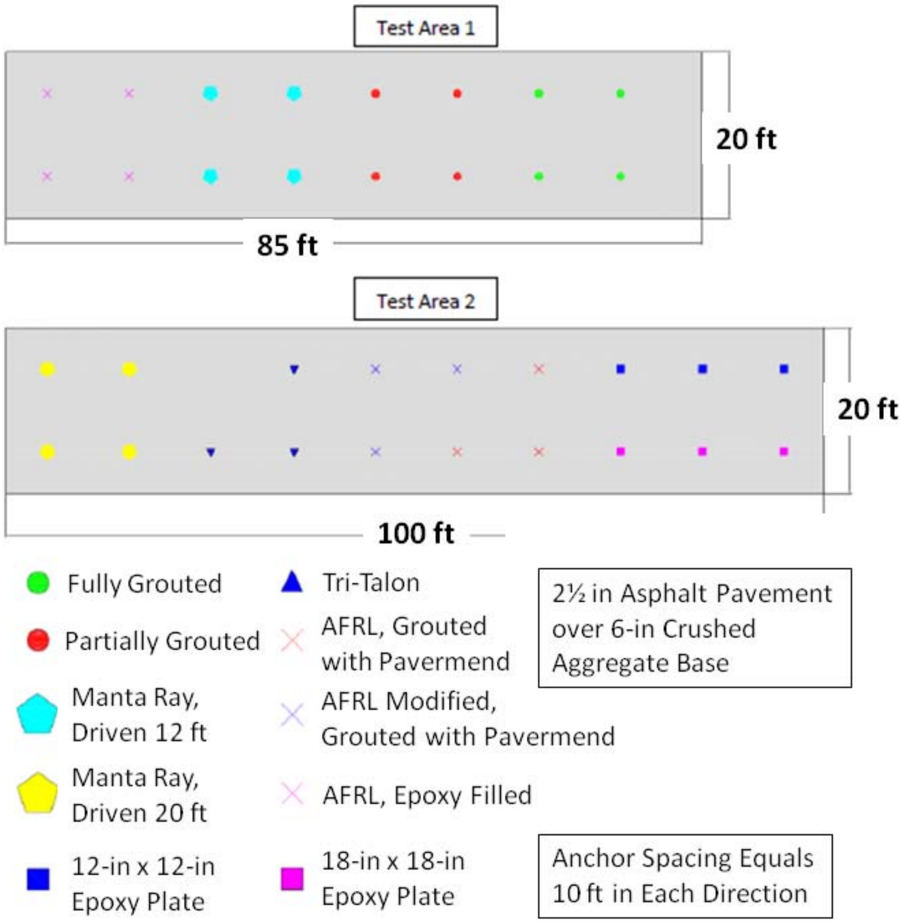


Figure 29. Silver Flag Test Layout

6.2.1.1. Fully Grouted Pier Installation Procedures

Four fully grouted piers were installed for load testing at the Silver Flag location. The following section describes the installation of these tie-downs. The process did not vary significantly by location. Location specific details are discussed, when necessary, in the appropriate sections of this report.

Pavement Coring. The HMA layer was cored using an 8-in-diameter core bit prior to augering to minimize the damage potential to the auger equipment. Figure 30 shows the coring and core area after extraction.

Figure 30. Pavement Coring Operations

Augering. Augering equipment was utilized to create a cavity for the fully grouted pier. The augering process consisted of multiple insertions, each insertion increasing the cavity's depth by approximately 2 ft. As the auger began to remove the sub-grade soil it was necessary to remove the auger and clear the paddles of spoil material. Figure 31 shows the augering process.

Water was encountered 4 ft below grade, raising concerns regarding the stability of the shaft. Ultimately, each shaft remained intact during the process. However, there was noticeable water intrusion into the cavity, which was an issue because grout is water sensitive and increasing the water–cement ratio could have significantly altered its performance.

Figure 31. Augering 8-in-diameter Hole for Fully Grouted Pier

Metal Tendon Insertion. Following the augering process, a 1-in-diameter, continuously threaded, metal tendon was placed in the shaft. To alleviate water intrusion concerns and ensure the rod achieved a proper bond with the concrete, the bottom 1 ft of the metal tendon was placed in a pre-fabricated concrete mold, forming a concrete plug. The form used for the concrete plug was a 6-in by 12-in cylinder form. The metal tendon was centered at the bottom of the form and concrete was placed. After this process was completed, the concrete was allowed to set before the rod and the accompanying concrete plug were inserted into the shaft.

Each metal tendon was 10 ft in length, guaranteeing that roughly 3 ft of the rod remained above the pavement surface after insertion. This was to allow for an attachment point to conduct load testing at the appropriate time. Ultimately, the metal rod was trimmed so the top was 4 to 5 in above the pavement surface. However, the threaded bar was not trimmed until after the grout placement process, as described in the next section.

Grout Placement. To expedite the installation and testing process, Pavemend 15.0® rapid-setting grout was utilized as a bonding agent. Pavemend is a water-activated, cementitious, self-leveling, structural repair mortar, with a working time of 7–9 min.

Pavemend is capable of achieving compressive strengths of 3,000 psi within two hours and 6,000 psi at 28 days[20]. The product can be applied in ambient temperatures ranging from 30 to 110° F, although the ideal water temperature for mixing is between 65 and 75° F. Water temperature above this range decreases set time, while water temperatures below this range increase set time.

The proper mixture procedure is to add one gal of water for each 45-lb bucket of Pavemend material, and to agitate with a paddle mixer for not less than 2½ min. The grout is ready for placement after a uniform mixture has been achieved. Due to the rapid setting times, Pavemend cannot be distributed with a concrete pump and is typically poured directly from the bucket to the appropriate location (Fig. 32).

Individual Pavemend buckets have a material yield of 0.42 ft^3. Each fully grouted pier cavity required between six and eight buckets of material to fill the shaft to the pavement surface. The grout material was agitated with a concrete vibrator to eliminate air voids and ensure consolidation. After vibrating, material was added as necessary to form a smooth surface with the adjacent pavement sections. Following placement, the rapid-setting cementitious material was allowed to set for a minimum of 24 hours before the anchor system was subjected to loading conditions.

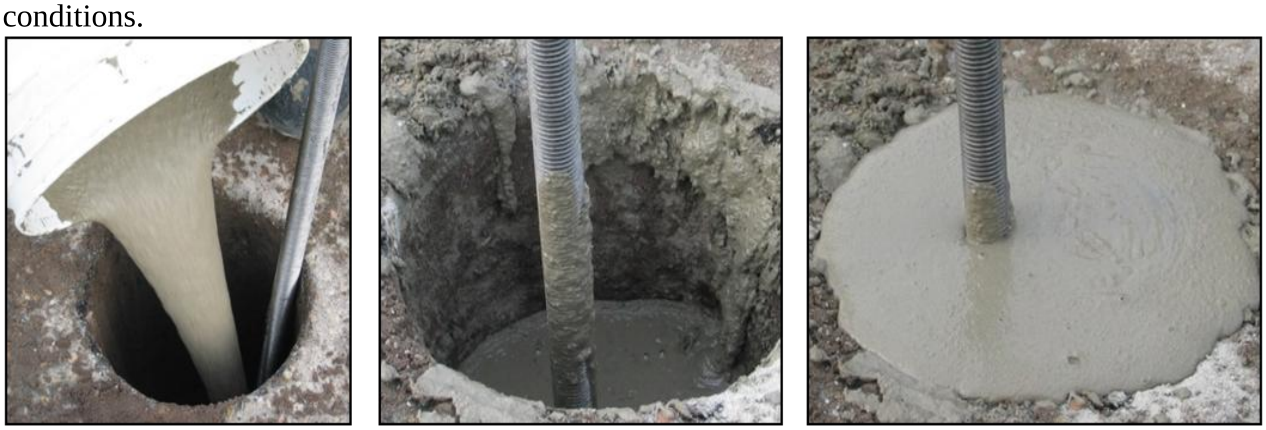


Figure 32. Rapid-Setting Grout Placement

6.2.1.2. Partially Grouted Pier Installation Procedures

Four partially grouted piers were installed for load testing at the Silver Flag Exercise Site. The following section describes the installation of these tie-downs. The process did not vary significantly by location. Location-specific details are discussed, when necessary, in the appropriate sections of this report.

Pavement Coring. The pavement coring procedure was the same as described previously.

Augering. The augering procedure was the same as described previously.

Metal Tendon Insertion. The metal tendon insertion procedure was the same as described previously.

Grout Placement. Grout Placement procedures for partially grouted piers varied significantly from those for fully grouted piers. After the hardened concrete plug was inserted into the augered shaft, the bottom 1 ft of the cavity was filled with grout. The remainder of the shaft was left un-grouted.

Backfilling. After the grout had been allowed time to set, most of the remaining cavity was filled with soil excavated during the augering process. Before backfilling, the soil was chemically stabilized with Portland cement. The stabilization process was simple in nature. Soil removed from the hole was spread out and allowed to dry for 2–3 hours. After the drying period, Portland cement was mixed into the soil with basic hand tools. Each shaft required 1½–2½ ft^3 of soil, mixed with 46 lbs of cement, to sufficiently backfill the cavity.

Sub-grade material was backfilled and compacted in 1-ft lifts. A small tamping rod was utilized to perform the compaction. The tamping rod was a fabricated device consisting of a hollow metal rod with a 2-in by 2-in by ¼ -in plate welded to the bottom. The tamper was small enough to fit between the threaded bar and the sidewall of the shaft. The cavity was backfilled to a height 1 ft below the pavement surface. Grout was then utilized to fill the remainder of the cavity until the grout level was even with the adjacent pavement surface. After allowing sufficient time for the Pavemend to set, the top of the threaded rod was trimmed 4–5 in above the pavement surface. This allowed attaching the anchor puller to the test specimen (Fig. 33).

1) Trim extruded portion of threaded rod

2) Attach Crosby lifting hoist

3) Specimen prepared for pull-out testing

Figure 33. Typical Attachment Set-up

6.2.1.3. Manta Ray Earth Anchor Installation

Eight Manta Ray anchors were installed at the Silver Flag Exercise Site. Four anchors were installed to a depth of 12 ft, and the remaining four to a depth of 20 ft. Manta Ray anchors are driven plate anchors that mechanically attach to a metal tendon. To drive the plate to the required depth, removable drive steel was inserted into the Manta Ray anchor and the entire system was hydraulically driven into the substrate material. After achieving the appropriate driving depth, the drive steel was removed. A load locking device was then attached to the metal tendon to exert a tensile force on the tendon, forcing the plate anchor to rotate back towards the surface.

The load locker is a hydraulically powered mechanism designed to force the Manta Ray to rotate until the orientation of the anchor is parallel to the ground surface, as opposed to its perpendicular driving orientation. It was often necessary to pull the Manta Ray 2 ft or more before the anchor locked in.

Theoretically, load lockers also served as a proof load. They exert pressure until the Manta Ray anchor is forced to rotate and lock into a final orientation rotated 90° from the driving position. The load locking device exerted a tensile force on the metal tendon until refusal was reached and the anchoring system would not pull out of the ground anymore. Load lockers were available in different classes, delineated by loading capacity. The typical Manta Ray load locker had a 20,000-lb capacity, although higher-capacity load lockers were available. A dial gauge situated on the device allowed the operator to read and adjust the load and loading rate. In theory, if the load locker reached its full capacity and the Manta Ray anchor had stopped moving, two conclusions could be drawn. The plate was fully locked in and the installed tie-down was capable of a pull-out resistance equal to the capacity of the load locking equipment.

The following sections detail the Manta Ray installation process. Installation procedures at each location were similar. Location-specific details are summarized as appropriate.

Coring. The coring procedure was the same as previously described. The only difference was that 12-in-diameter cores were extracted for Manta Ray Installation.

Metal Tendon Attachment. Metal tendons, in the form of continuously threaded bar, were attached to transmit the tensile loading forces to the Manta Ray anchor. Manta Rays were capable of accepting 1-in-diameter threaded bar. To attach, the rod was screwed into a hinge shackle on the anchor. The hinge shackle was custom threaded to the purchaser's request. The hinge allowed the plate to rotate into place during the load locking process. However, the shackle was open on the bottom and it was possible to thread the rod completely through the shackle. This was problematic because the rod could possibly bind the anchor plate and prevent proper rotation of the Manta Ray. To mitigate this concern, after the rod was threaded to the bottom of the hinge shackle, a lock nut was employed to eliminate further threading of the rod during the driving process (Fig. 34).

Although threaded rod was available in segments of 12 ft or more it was not possible to utilize a single piece of rod. Shorter segments, attached with heavy duty couplers, were utilized to achieve the requisite length.

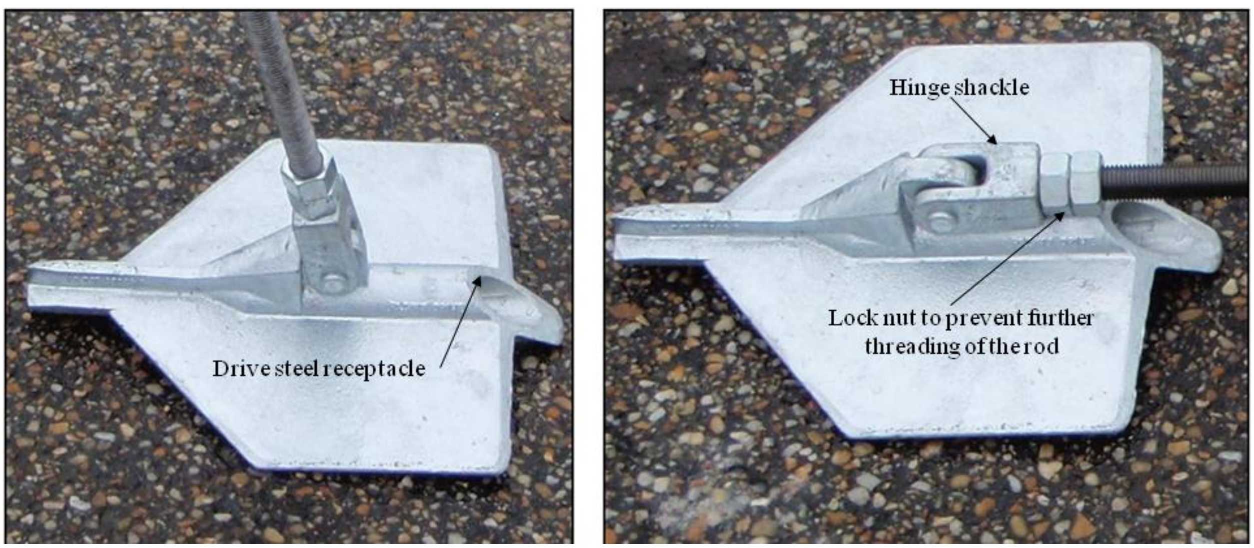


Figure 34. Manta Ray Anchor Attached to Threaded Rod

Manta Ray Driving. Manta Ray anchors were mechanically driven with a hydraulic powered mounted breaker, which was attached to a skid steer (Fig. 35). The mounted breaker was equipped with a blunt impactor, which was a necessary component to drive the Manta Ray tie-downs. The blunt impactor mated to the drive steel components, which in turn mated to the Manta Ray earth anchor. Essentially, the blunt impactor hammered the drive steel, driving the anchor system into the soil.

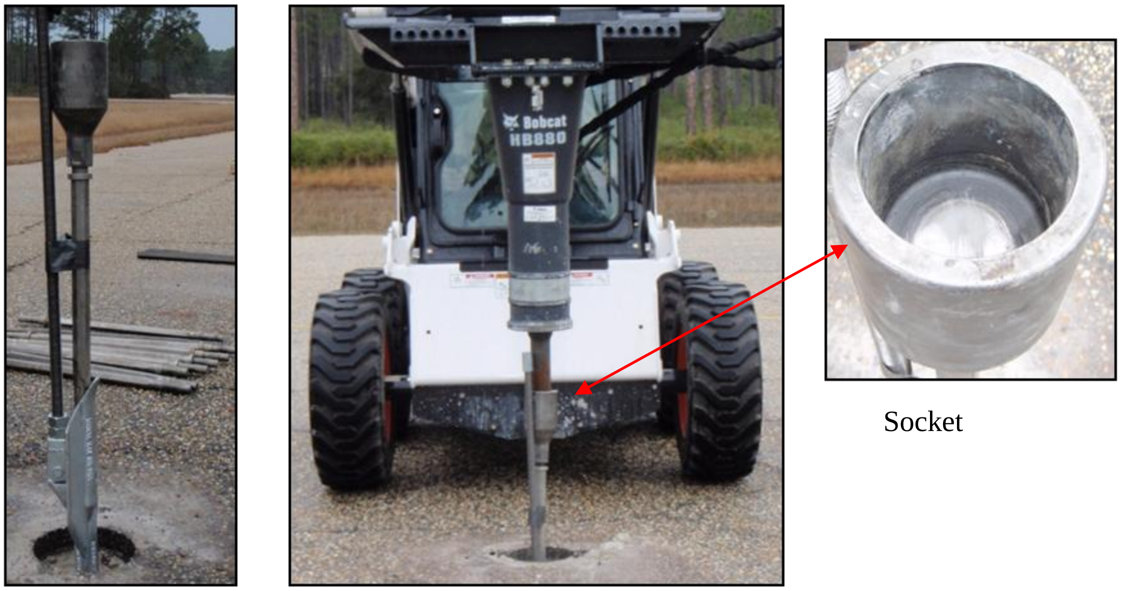


Figure 35. Manta Ray Driving Operation

A socket adapter was necessary to mate the blunt hammer to the drive steel components. The socket adapter threaded to the top of the drive steel, providing a female socket for the blunt tool. Numerous socket adapters are available to accommodate blunt impact tips of various diameters.

The requisite drive steel components necessary to perform an installation were the drive tip, drive steel couplers, drive steel extensions and the socket adapter. One end of the drive tip inserted into a female receptacle on the Manta Ray (Fig. 34). The opposing end of the drive tip was threaded to allow the heavy-duty couplers to attach to the drive tip itself and additional drive extensions. Drive extensions were threaded on each end to allow for attachment of additional drive steel extension pieces. The diameter of drive steel tips and extensions was 1¼ in. Drive tips were available in 2½-ft, 6-ft, and 8-ft lengths. Extensions were available in 33-in, 6-ft, and 8-ft lengths. AFRL testing utilized the 2½-ft drive tip and 33-in drive extensions. Drive steel components, including the socket adapter, were purchased from the Manta Ray manufacturer.

Each segment was driven vertically into the ground until the bottom portion of the uppermost coupler and the top of the threaded rod, were even with the pavement surface. The blunt hammer was then disengaged from the socket adapter, and the socket adapter was removed from the uppermost coupler. The uppermost coupler remained attached to the extension piece in the ground. An additional extension piece was then attached to the top of that coupler. Next, a separate coupler was attached to the top extension piece and the socket adapter was threaded into place on the top side of the top collar. Finally, the blunt hammer was re-engaged with the socket adapter and the process was repeated until the appropriate depth was reached. It is imperative to note that 2 to 3 ft of the threaded rod remained above the pavement surface after driving operations ceased. This was necessary for load locking.

It was important to ensure the skid steer operator maintained vertical alignment during the driving process. The impactor was at risk to fracture the drive steel if the operator did not keep the blunt hammer and drive steel aligned, which was a danger to personnel present in the area. This process required an experienced operator and a signal guide.

After achieving the proper installation depth, the drive steel was removed. This was accomplished by attaching a chain to the drive steel and pulling it out with a loader. To facilitate the rigging procedure the socket adapter was not removed from the top of the drive steel after disengaging the blunt hammer. The socket adapter provided an excellent choking point for the chain to lift the drive steel.

Load Locking. Load locking the anchor was necessary to rotate and fully engage the anchor. The load locker was placed over the extruded portion of all thread and engaged. Figure 36 demonstrates the load-locking process. The load locker had an 8-in-stroke and typically needed to be re-positioned several times with each anchor. It usually required between 1½ and 2½ ft of vertical movement to fully engage and rotate the anchor. The process was completed when the plate refused to extract from the ground any further. Theoretically, load locking the anchor was a proof test of the tie-down's pull-out capacity.

Grouting. A void was created by the driving process and removal of the drive steel. After fully engaging the anchor it was important to maintain pressure with the load locker until the cavity was grouted, and the grout had time to set (Fig. 37). There was a possibility the plate may have rotated back towards its driving orientation if the pressure was released before the grout set.

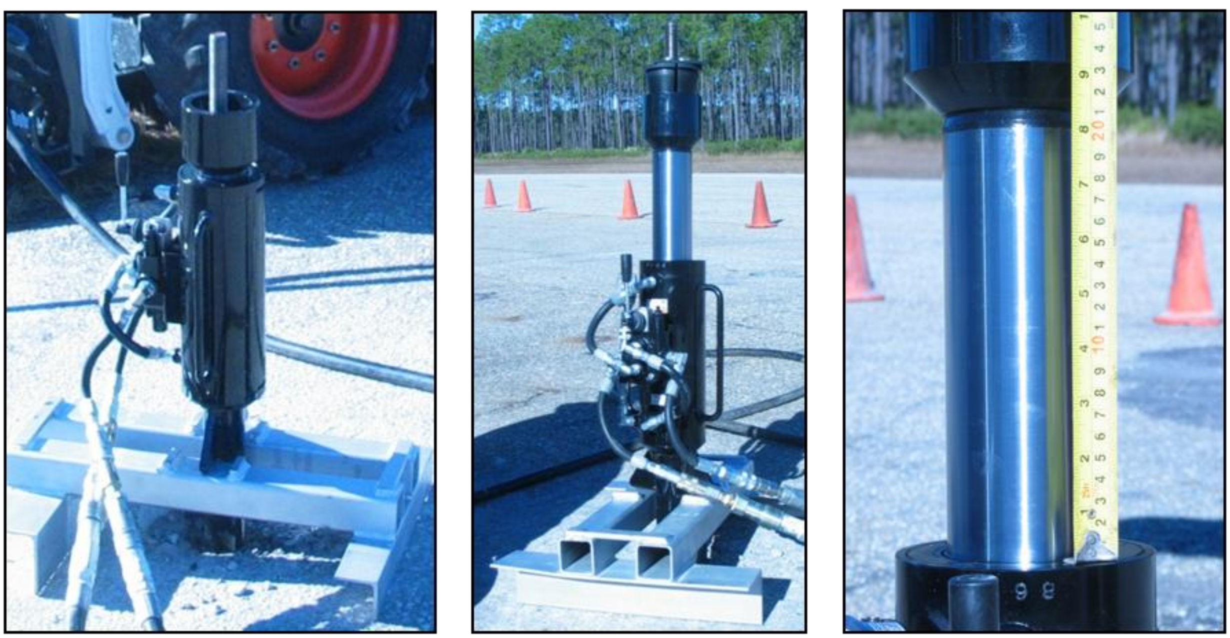


Figure 36. Manta Ray Load Locking Operation

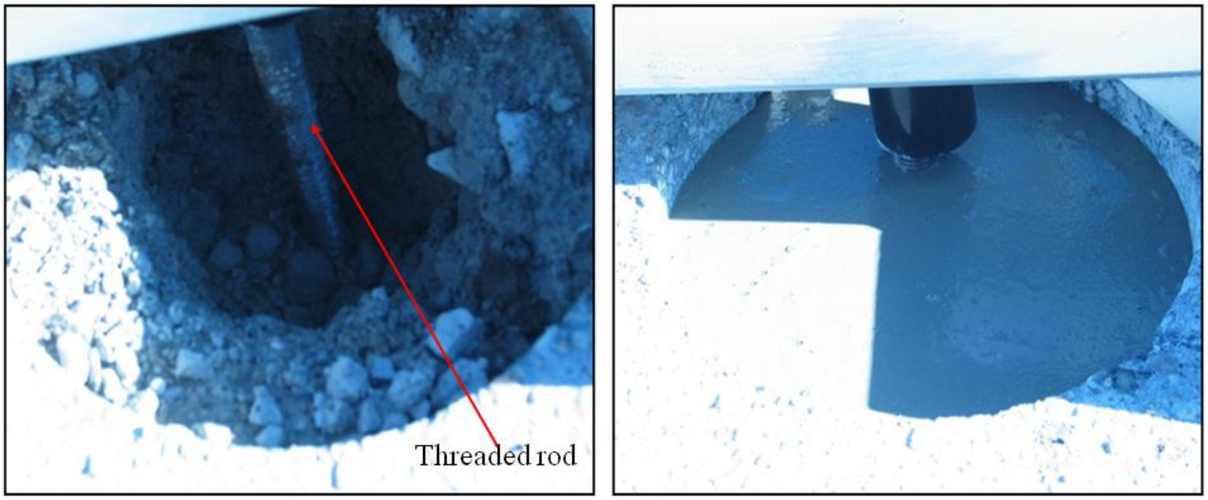


Figure 37. Anchor Grouting after Load Locking Manta Ray Tie-down, before (left) and after (right) Grouting

The cavity dimensions were variable. Normally, the shaft diameter measured 3–4 in. However, the void depth varied from 1½ to 10 ft, depending on the particular anchor. Rapid-setting grout was placed in each shaft and vibrated to completely plug the cavity and provide an even surface with the surrounding pavement.

6.2.1.4. AFRL Epoxy Anchor Installation

Three single epoxy anchors were installed using LiquidRoc 500 epoxy and load tested at the Silver Flag Exercise Site. Their installation required a minimal logistical footprint, which is desirable in contingency environments.

LiquidRoc 500 is a two-part epoxy specifically designed as a concrete adhesive. It is packaged in a single 8.5-fl oz tube and designed to be dispensed with a standard caulking gun. Cure times range from 6 to 24 hours, depending on the concrete temperature(21). The product is not recommended for temperatures below 40 °F. AFRL testing was conducted in a section of thin asphalt (2–3 in thick), underlain by a 6-in-thick base course layer and a silty sand subgrade. The manufacturer does not recommend the application of LiquidRoc 500 in these conditions.

Drilling. The initial installation step was to drill a 2-in-diameter, 18-in-deep hole through the asphalt and base course layer. After drilling, it was necessary to use a shop vacuum to remove loose soil from the drilled hole, being cautious not to increase the hole depth during the vacuuming process.

Under-cutting. Under-cutting was the process of creating a soil bulb, thus creating a larger failure zone. A specially designed tool, an under-reamer, was utilized to perform the under-cutting operation. The under-reaming tool was essentially identical to the under-reaming tool designed by Williams Form Engineering, for testing conducted by ERDC(18). After drilling and vacuuming the hole, the under-reamer tool was inserted into the shaft. The under-reaming tool is exhibited in Figure 38. Following the under-reaming operation it was necessary to again vacuum the shaft, removing loose soil while being careful to maintain the proper cavity depth. Figure 39 shows the under-cut section created by the under-cutting tool.

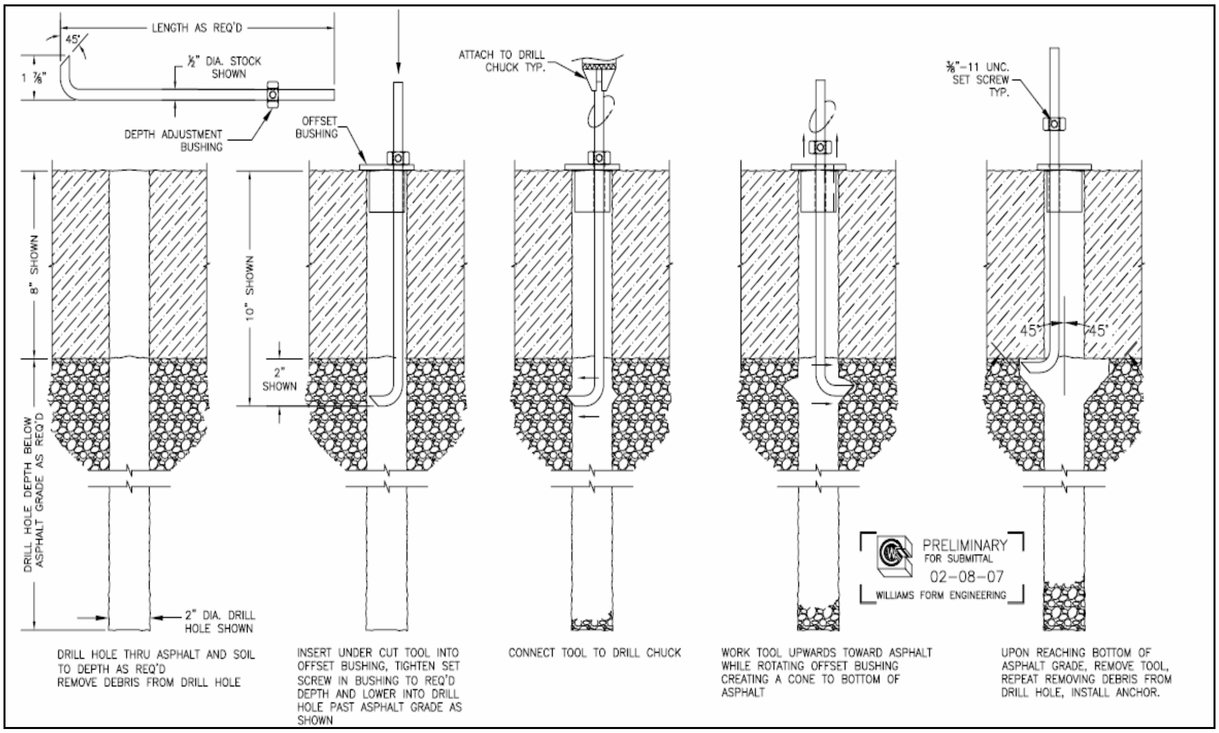


Figure 38. Under-cutting Operation Schematic[18]

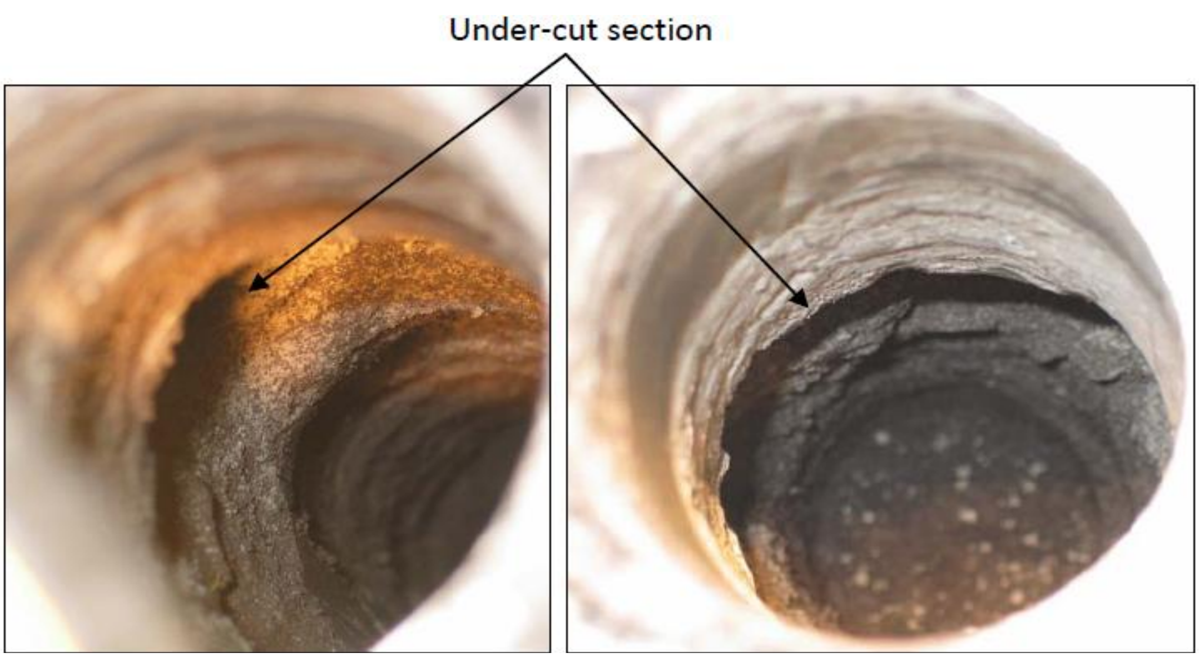


Figure 39. Under-cut Section of Drilled Shaft

Epoxy Application. Epoxy was dispensed into the hole after the drilling and under-reaming process. Dispensing was accomplished with the use of a battery-powered caulk gun. The battery-powered caulk gun significantly decreased the time needed to fill the cavity with epoxy. This was imperative because to adequately fill each hole required six to eight epoxy tubes. Performing this operation with a hand-powered dispenser would have increased the likelihood that the initial epoxy tubes would begin to set before the tie-down was inserted into the shaft.

Epoxy application was ceased after filling the shaft to within ½ in of the pavement surface. The modified Hilti anchor bolt was then inserted into hole. It is important to note LiquidRoc 500 set times were appreciably influenced by the surface temperature. Additionally, the ambient temperature notably affected the viscosity of the epoxy. Higher ambient temperatures resulted in a decreased viscosity.

6.2.1.5. AFRL Epoxy Anchor Plate Installation

Six anchor plates were installed; three 12 in by 12 in, and three 18 in by 18 in. The installation concept was very similar to that of the single epoxy tie-downs. The plate was utilized as a template to accurately mark epoxy anchor locations in the pavement. Anchor holes were drilled in the same manner as described previously. After drilling and reaming the holes, the plate was placed on the pavement surface, ensuring the plate holes lined up with the anchor holes on the pavement. Each cavity was then filled with epoxy and the modified wedge bolts were inserted through the plate (Fig. 40). A structural washer under the bolt head ensured the epoxy tie-down did not completely slide through the anchor plate.

Figure 40. Installed Epoxy Anchor Plate

6.2.1.6. Tri-Talon Anchor Installation

Tri-Talon anchor installation required simple, readily available field implements. The tie-down was essentially a ¾- in-diameter segment of continuously threaded rod with a talon attachment mechanically connected to the bottom of the rod. A coupler attached to the top of the rod served as a tie-down point. The coupler was removed during the installation process, and re-attached after driving the Tri-Talon to the appropriate depth.

Anchor installation required a 3-in-diameter hole, cored or drilled to a depth of 16–18 in into the base course and sub-grade material. The rod and talon attachment (talons in vertical position) was inserted into the drilled cavity. A drive shaft, consisting of a hollow metal tube, was then inserted into the cavity. The diameter of the drive steel allowed for it to slide easily over the threaded rod, but not over the talons. As force was applied to the drive steel it compelled the tie-down into the substrate, at the same time forcing the talons into a horizontal position. In essence, the talons created a soil bulb as they deployed, expanding the failure surface. This expanded failure surface theoretically increased the amount of sub-grade material resisting anchor pull-out. After driving the anchor to the requisite depth, the cavity was grouted. It was important to seal the top-side of the coupler to eliminate grout intrusion and ensure the anchoring bolt could properly thread into the coupler. The installation process is demonstrated in Figure 41.

Figure 41. Tri-Talon Anchor Installation

6.2.1.7. AFRL Rapid Set Grout Anchor Installation

Three of these particular mooring point installations were performed by AFRL. This anchor installation procedure was identical to that of the epoxy anchors, except that rapid setting grout was utilized as a bonding agent. The object-tive of this testing was to determine if the grout provided the same pull-out capacity as the epoxy.

This option offered an abbreviated installation timeline, in comparison to the epoxy tie-down. One 45-lb bucket of Pavemend had a yield of 0.42 ft^3, which provided enough material to install six to ten AFRL mooring points. The time required to thoroughly mix and place one bucket of grout was significantly less than the time required to dispense an adequate amount of epoxy to fill the same number of anchors

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6.2.1.8. Modified AFRL Rapid Set Grout Anchor Installation

Three modified AFRL rapid set grout anchors were installed at the Silver Flag Exercise Site. As previously noted, the installation procedure was identical to the AFRL rapid set tie-downs.

6.2.2. Seguin Auxiliary Airfield Testing

Seguin Auxiliary Airfield, constructed in 1944, is located 25 miles east of San Antonio, TX. The surrounding area is characterized by pastoral homesteads and low-grade hilly areas. Seguin's climate is categorized by warm and humid summers, and cool to moderate winters. The elevation is 532 ft above sea-level. Average yearly precipitation is 30.4 in, falling in the form of rain and snow. Rainfall is fairly evenly distributed throughout the year, although heavy rains are common from April through September. Hurricanes inbound from the Gulf Coast pose wind threats, with the greatest possibility of occurrence in September. Historically, Seguin-area tornado activity is slightly below the Texas state average, but 60 percent greater than the U.S. average. Tornado-related wind gusts of 158 knots have been recorded in the area.

Testing was conducted in an abandoned section of the parking apron. The flexible pavement surface was in very poor condition and measured 1–2 in-thick. The pavement surface was situated on an 18-in-thick, compacted caliche base course layer with a California bearing ratio (CBR) of 15. Caliche is best described as a hardened deposit of calcium carbonate. The calcium carbonate acts as a binder, cementing together mineral aggregate deposits. The subgrade material comprised a silty clay with a CBR of 10. Water was encountered 5–6 ft below grade. Anchor installation at Seguin included the following tie-downs:

a. Fully grouted piers
b. Partially grouted piers
c. MR-SR soil anchors
d. AFRL epoxied anchors
e. AFRL epoxied anchor plates

Installation procedures for each anchor and load testing results are presented in detail in subsequent sections of this report. The Seguin test layout is shown in Figure 42.

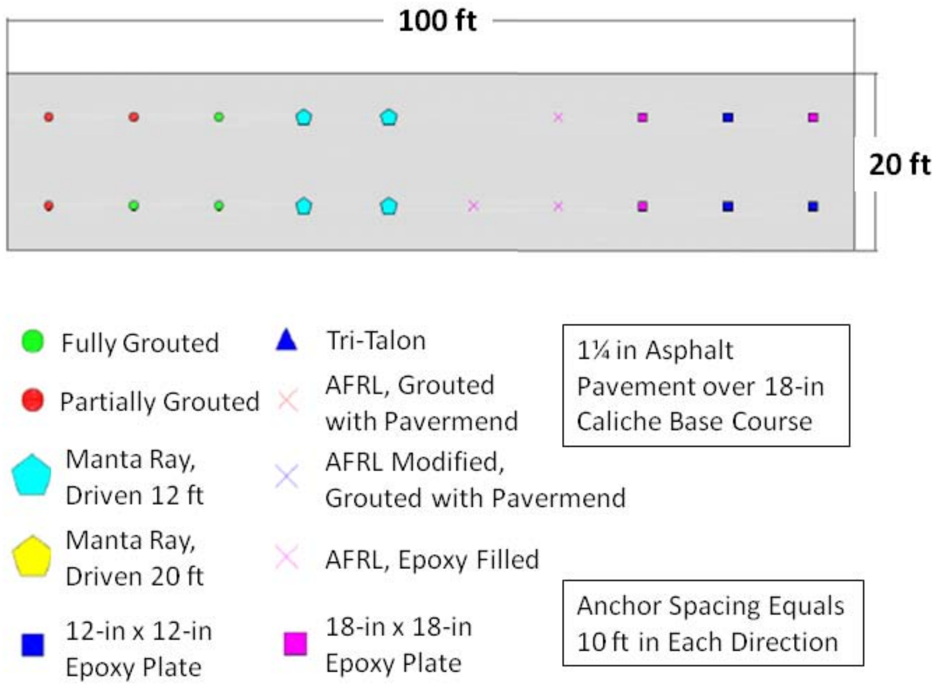

Figure 42. Seguin Test Layout

6.2.2.1. Fully Grouted Pier Installation Procedures

Three fully grouted piers were installed in the same manner as described previously. The augering process was more difficult due to the presence of clay, which adhered to the auger paddles. Six to eight Pavemend buckets were used to fully grout the cavity.

6.2.2.2. Partially Grouted Pier Installation Procedures

Three partially grouted piers were installed in the same manner described previously. The cohesive nature of clay made it difficult to adequately mix the Portland cement with the excavated sub-grade. After thoroughly mixing the clay and cement, the treated material was used to backfill the cavity in 1-ft lifts. The top 1 ft of the shaft was filled with grout until it was even with the pavement surface.

6.2.2.3. Manta Ray Earth Anchor Installation

Four Manta Ray Anchors were installed in the same manner described previously. The original test matrix specified three installation depths of 12 ft and one installation depth of 20 ft. However, each anchor reached refusal at a depth of 12 ft and the testing was ceased. After load locking, three of the anchors were filled with grout and one left ungrouted.

6.2.2.4. AFRL Epoxy Anchor Installation

Three single epoxy anchors were installed in the same manner as described previously. However, the thickness of the base course layer required under-cutting in the compacted base course, as opposed to directly beneath the base course. Thus, the bulb was located in the compacted material.

6.2.2.5. AFRL Epoxy Anchor Plate Installation

Six anchor plates were installed in the same manner as described previously.

6.2.3. Avon Park Air Force Range Testing

Avon Park Air Force Range, established in 1942, is a 106,000-acre bombing and gunnery range. The range is located in central Florida at the confluence of Okeechobee, Polk and Highlands counties. Avon Park is approximately 100 miles east-southeast of MacDill AFB, FL. During World War II, the site was known as Avon Park Army Air Field and was used as a training base for B-17 aircraft crews for air-to-ground bombing. Prior to its use by the military, most of the land was unimproved pasture and swampland.

This part of Florida is characterized by a water table at or near the surface for the majority of the year. The land is irregular due to the dissolution of its limestone bedrock by acidic ground water. This causes caverns, sinkholes, pinnacles, solution pipes and a honeycomb-structure of voids in the limestone.

The climate is classified as sub-tropical because of its low latitude and high relative humidity levels. Summer conditions are typically hot and humid, while winter months are relatively mild. The rainy season extends from May through September. Average annual precipitation is 53.8 in. During the rainy season afternoon thundershowers are an almost daily occurrence and it is not uncommon for portions of the base to briefly flood. The site elevation is 156 ft above sea level. Peak wind gusts of 55 knots have been measured at the Avon Park Air Force Range.

Testing was conducted in an abandoned section of the airfield. The flexible pavement surface was in very poor condition and thickness measured 1–2 in. The pavement surface was situated on a 6-in-thick cement-stabilized base course with a CBR of 100. The base course appeared to be native mineral aggregate thoroughly mixed with cement. Several 3-in-diameter by 6-in-deep core samples (Fig. 43) were extracted from the base course material and subjected to compressive testing in accordance with ASTM D1633, *Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders*. The cylinders' mean compressive strength measured 2,800 psi. The subbase and subgrade layers consisted of poorly graded silty sand, each with a CBR of 15. Water was encountered 5–6 ft below grade.

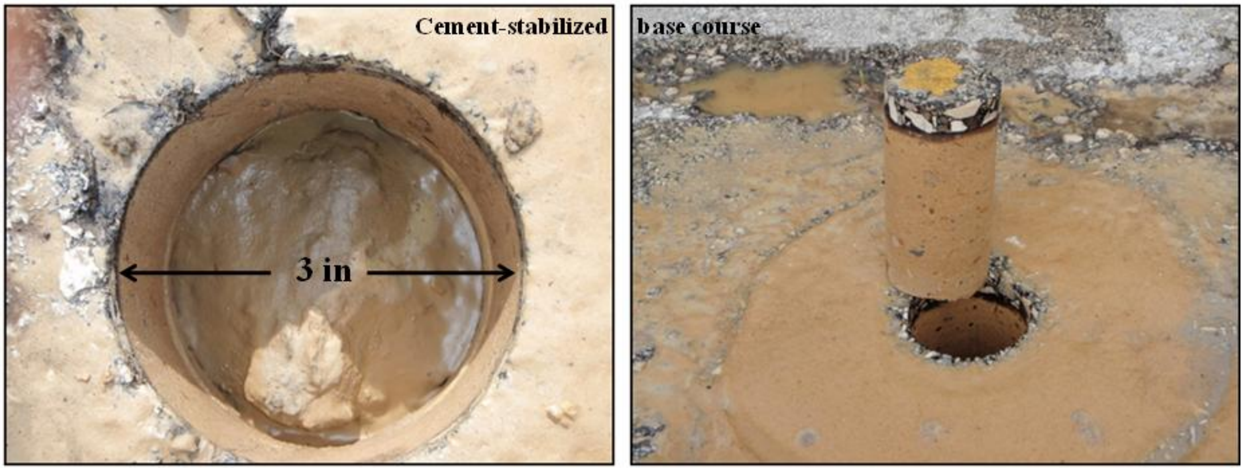

Figure 43. Core Sample and Hole Cut in Cement-Stabilized Base Course, (Avon Park)

Anchor installation at Avon Park included the following tie-downs:

a. Fully grouted piers
b. Partially grouted piers
c. MR-SR soil anchors
d. AFRL epoxied anchors
e. AFRL epoxied anchor plates
f. AFRL rapid-set anchors
g. Tri-talon anchors

Installation procedures for each anchor and load testing results are presented in detail in subsequent sections of this report. The Avon Park test layout is provided in Figure 44.

6.2.3.1. Fully Grouted Pier Installation

Three fully grouted piers were installed in the same manner as previously described.

6.2.3.2. Partially Grouted Pier Installation

Three partially grouted piers were installed in the same manner as previously described.

6.2.3.3. Manta Ray Earth Anchor Installation

Four Manta Ray Anchors were installed in the same manner as previously described. Three tie-downs were driven to a depth of 12 ft and one was driven to a depth of 20 ft.

6.2.3.4. AFRL Epoxy Anchor Installation

Three single epoxy anchors were installed in the same manner as previously described.

6.2.3.5. AFRL Epoxy Anchor Plate Installation

Six anchor plates were installed in the same manner as previously described. One of the large anchor plates was grouted with Pavemend, as opposed to LiquidRoc 500. That specific plate is notated in the data.

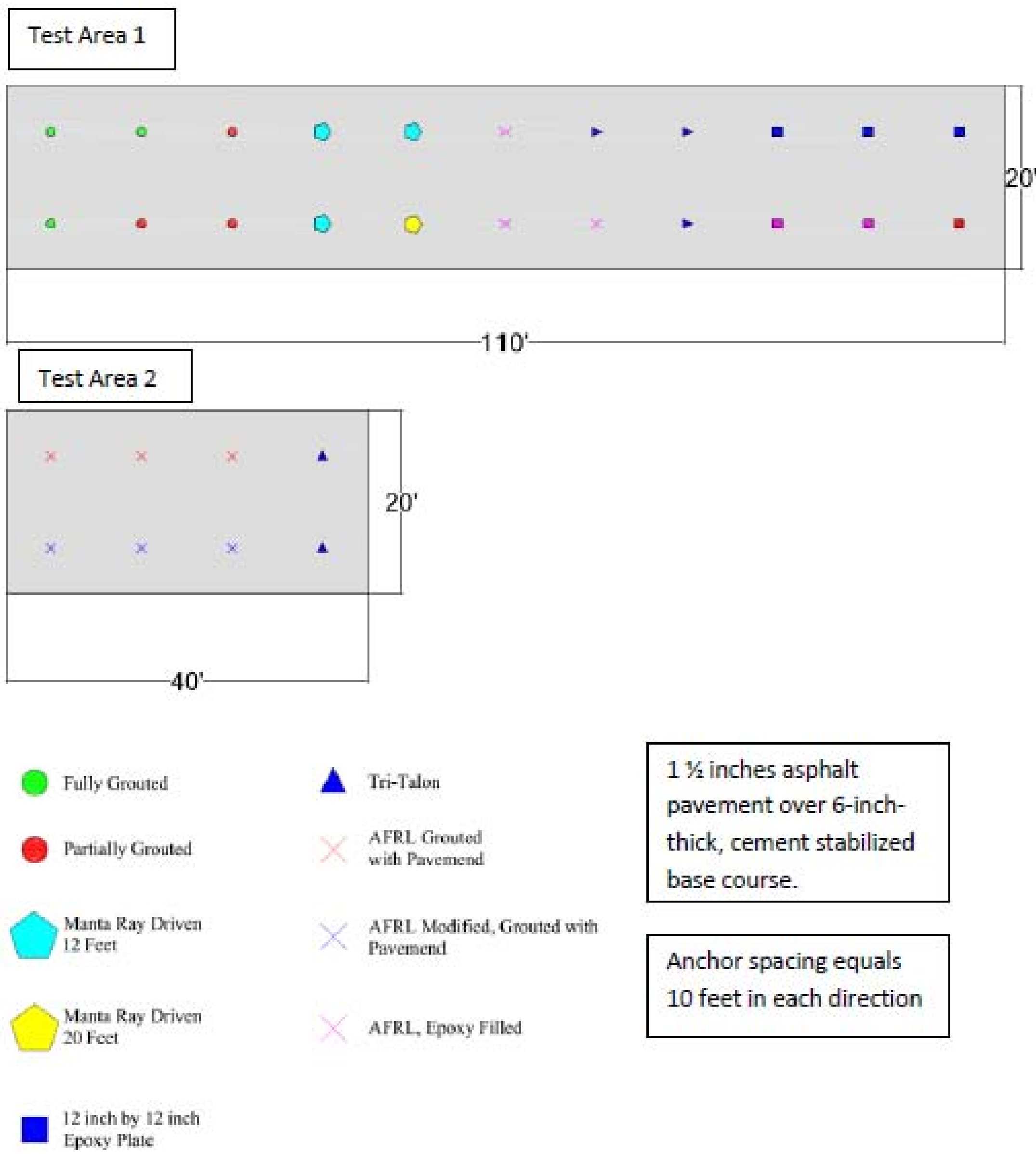

Figure 44. Avon Park Test Layout

6.3. Testing Results

Anchor testing results are detailed in the following section. Unless noted, each anchor was subjected to identical, static loading conditions. The loading process consisted of applying an initial 6,500-lb tensile force for a period of 60 sec. This initial load was increased by an additional 3,250 lbs in 60-sec intervals until testing operations were terminated due to one of the following events: 1) significant upheaval of the flexible pavement system, 2) bolt extraction in excess of 2 in, 3) refusal of the tie-down to accept additional loading demands. Henceforth, this particular loading schedule is referred to as *standard loading conditions*.

During the testing process, the assumed magnitude of the applied load was based on real-time pressure gage readouts supplied by the hydraulic pump. The pump-powered ram had an effective cylinder area of 6.49 in^2 and applied force through hydraulic pressure, which was regulated by a valve leading into the pump.

The performance of each individual anchoring system is discussed, and load-versus-deflection graphs are provided when appropriate. A comprehensive set of load/deflection graphs has been included in Appendices E, F, and G of this report. The graphs illustrate the applied force and the deflection readings obtained from the testing process. The displacement is represented by the blue curve, and the applied load by the red curve. Load and deflection increases are evidenced in the positive-slope portions of the curves. Negative-slope segments of the load curve illustrate a) termination of the testing process, or b) a slight bleedoff due to the inability of the anchoring system to maintain the full magnitude of the load. The negative-slope segments of the deflection curve depict a decrease in displacement. Deflection data were recorded for the duration of the testing process; thus the final displacement values were obtained after the load was removed. A slight amount of initial deflection was likely due to seating of the anchor puller's lifting and connection components. This initial seating deflection is difficult to quantify, but is possibly responsible for 1/8 to ¼ in of the recorded displacement.

Figure 45 is a typical load versus deflection graph, demonstrating a fully grouted anchor's response to the previously discussed standard loading conditions. Select tie-downs were loaded in a manner not consistent with standard loading conditions. For example, an individual anchor may have been initially loaded to 19,000 lbs for 10 sec and then unloaded. Within this report, those examples are notated and discussed.

The load-and-deflection graph functions represent a best-fit curve of the load cell and deflection gage data. The peak load and deflection values are illustrated by the high points on the respective functions. On some occasions, load and/or deflection data did not transfer to the data acquisition system. In these instances, the reported peak load is based on the peak pressure load recorded from the pressure gage during the testing process. These examples are notated within this report. Deflection estimates have not been provided in the absence of displacement data readings.

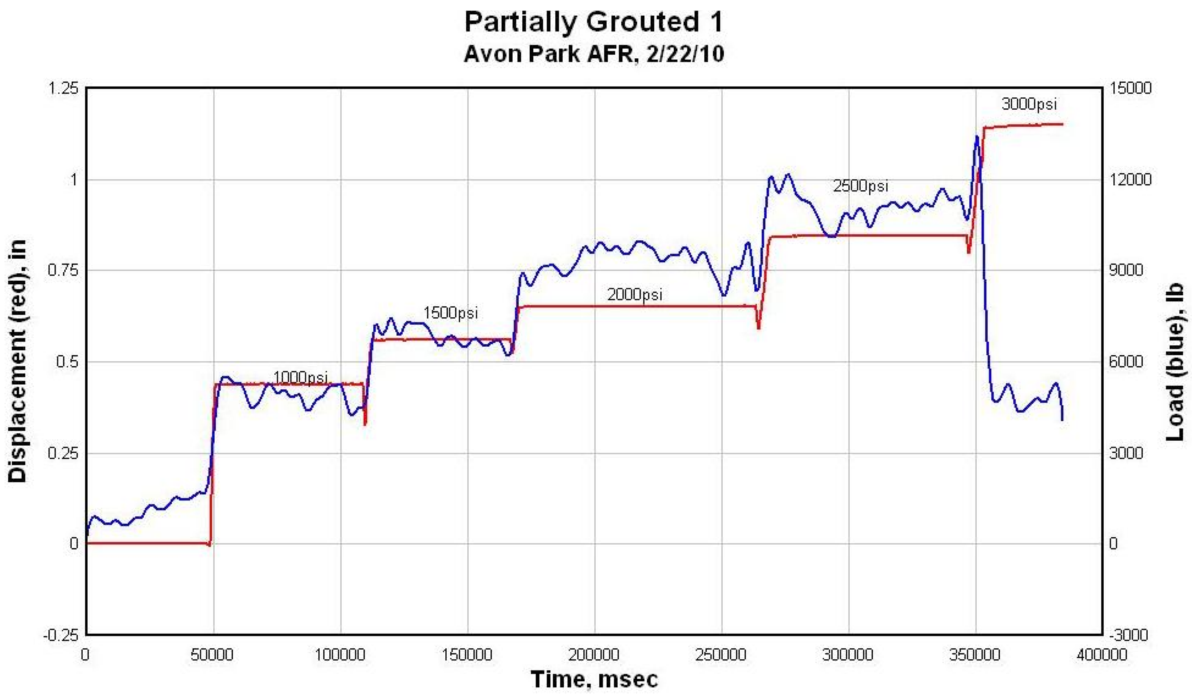

Figure 45. Example of a Typical Load–Deflection Graph. Anchor Subjected to Standard Loading Conditions.

6.3.1. Fully Grouted Piers

6.3.1.1. Silver Flag Exercise Site

Four fully grouted piers were installed and load tested at the Silver Flag site. Each pier was subjected to standard loading conditions. Ultimate uplift capacities ranged from a minimum of 7,350 lbs to a maximum of 14,480 lbs. The mean pull-out capacity was 11,050 lbs.

6.3.1.2. Seguin Auxiliary Airfield

Three fully grouted piers were installed and load tested at the Seguin Auxiliary Airfield. Each pier was subjected to standard loading conditions. Ultimate uplift capacities ranged from a minimum of 22,510 lbs to a maximum of 29,990 lbs. The mean pull-out capacity was 26,050 lbs.

6.3.1.3. Avon Park Air Force Range

Three fully grouted piers were installed and load tested at the Avon Park Range. FG-1 and FG-2 were loaded according to normal loading conditions. However, after termination of the initial testing procedure, both anchors were re-loaded. Pre-test and post-test pictures from FG-1 are shown in Figure 46. The loading and deflection functions for FG-1 are presented in Figures 47a and 47b. The objective of the reload was to determine if the mooring points could withstand a secondary load of approximately 20,000 lbs for a duration of 10 sec.

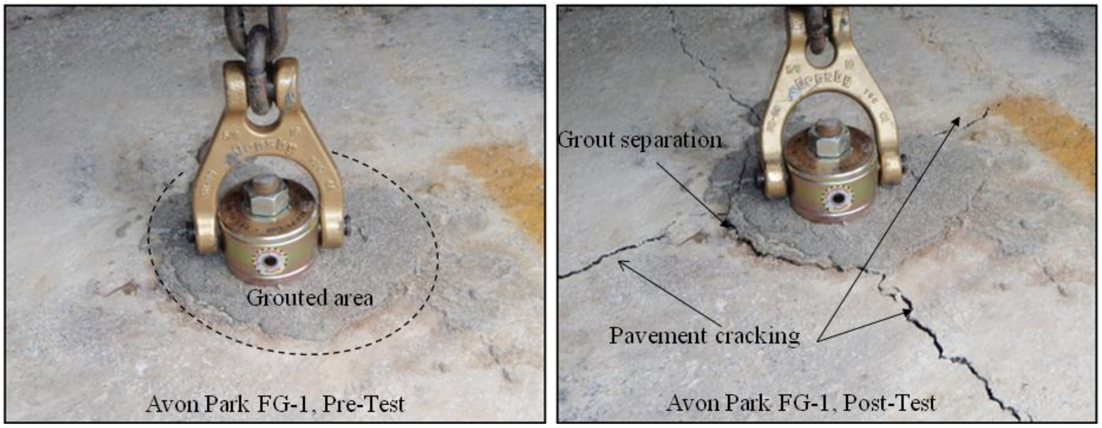

Figure 46. Avon Park FG-1, Pre-Test and Post-Test

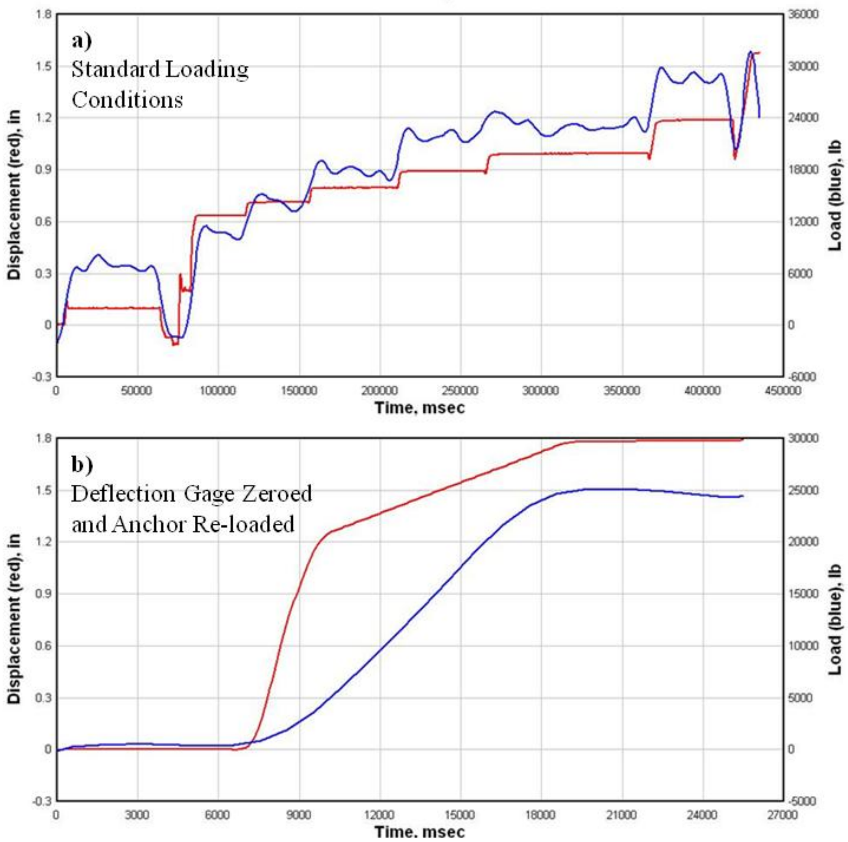

Figure 47. Avon Park FG-1 Tested a) at Standard Conditions, and b) after 10-t Loading

FG-3 was not loaded according to standard loading conditions. An initial 20,000-lb load was applied for 10 sec. Following the initial load, the anchor was re-loaded under standard loading conditions. This loading is represented in Figure 48. Ultimate uplift capacities ranged from 23,090 lbs to 31,750 lbs. The mean pull-out capacity was 28,590 lbs.

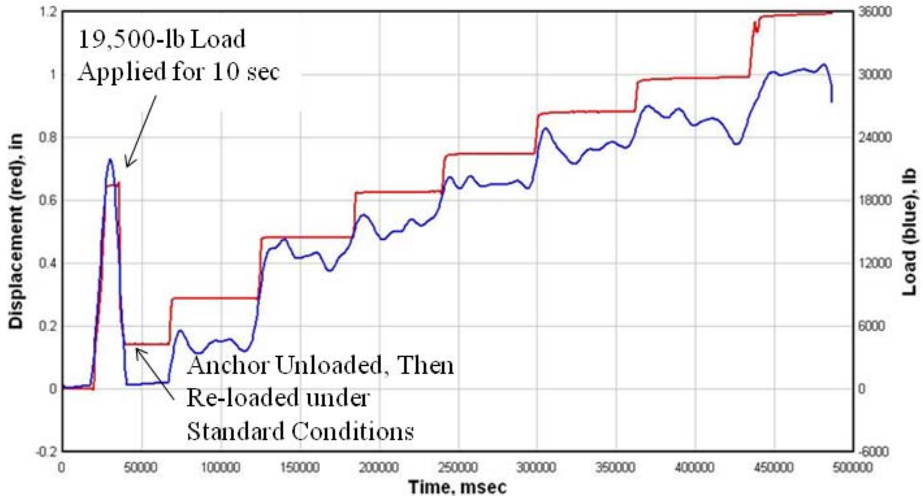

Figure 48. Avon Park FG-3

6.3.2. Partially Grouted Piers

6.3.2.1. Silver Flag Exercise Site

Four partially grouted piers were installed and load tested at the Silver Exercise Site. Each pier was subjected to standard loading conditions. Ultimate uplift capacities ranged from 4,830 lbs to 17,020 lbs. The mean pull-out capacity was 10,820 lbs. Figure 48 shows post-test pavement cracking from testing conducted on PG-2.

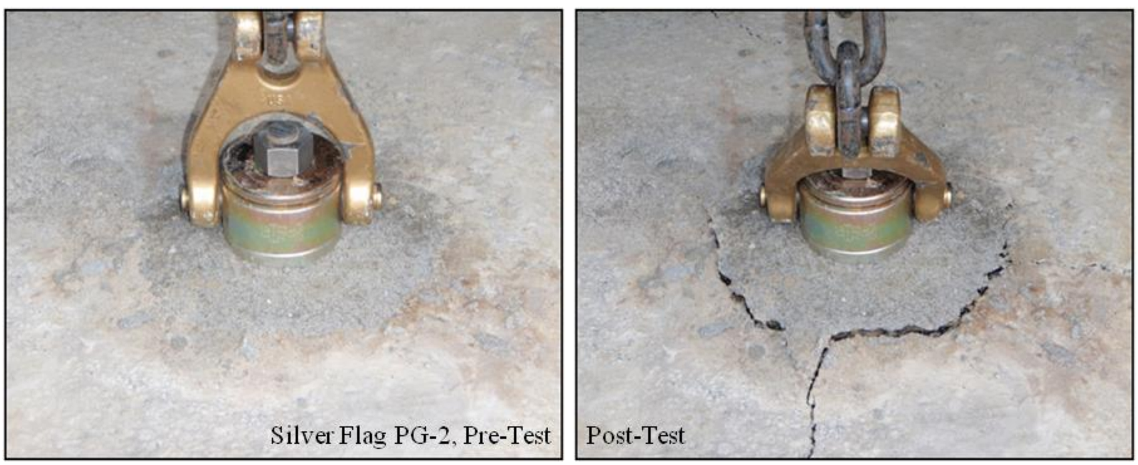

Figure 49. Silver Flag PG-2, Pre-test and Post-test

6.3.2.2. Seguin Auxiliary Airfield

Three partially grouted piers were installed and load tested using standard loading conditions at the Seguin Auxiliary Airfield. Ultimate uplift capacities ranged from 7,540 lbs to 14,920 lbs. The mean pull-out capacity was 10,210 lbs.

6.3.2.3. Avon Park Air Force Range

Three partially grouted piers were installed and load tested at the Avon Park Range. Each pier was subjected to standard loading conditions. Ultimate uplift capacities ranged from 13,310 lbs to 19,080 lbs. The mean pull-out capacity was 15,280 lbs.

6.3.3. Manta Ray MR-SR Earth Anchors

6.3.3.1. Silver Flag Exercise Site

Eight Manta Ray earth anchors were installed and load tested at the Silver Flag site. Four tie-downs were installed to a depth of 12 ft, and four to a depth of 20 ft.

12-ft Installation Depth. The four Manta Ray anchors installed to a depth of 12 ft were subjected to standard loading conditions. The load–deflection plot for anchor MR-2 is shown in Figure 50. Post-test pavement damage for MR-2 is illustrated in Figure 51. The tie-down labeled MR-3 exhibited a large displacement in comparison to the other three anchors in this group. It is likely that this anchor did not completely rotate into a horizontal orientation during the load locking process. Therefore, the anchor puller possibly performed the function of forcing the Manta Ray plate into its terminal orientation parallel with the pavement surface. The distance to complete load locking of the anchor exceeded the stroke of the ram. Figure 52 displays the extraction of the top portion of the MR-3 threaded rod.

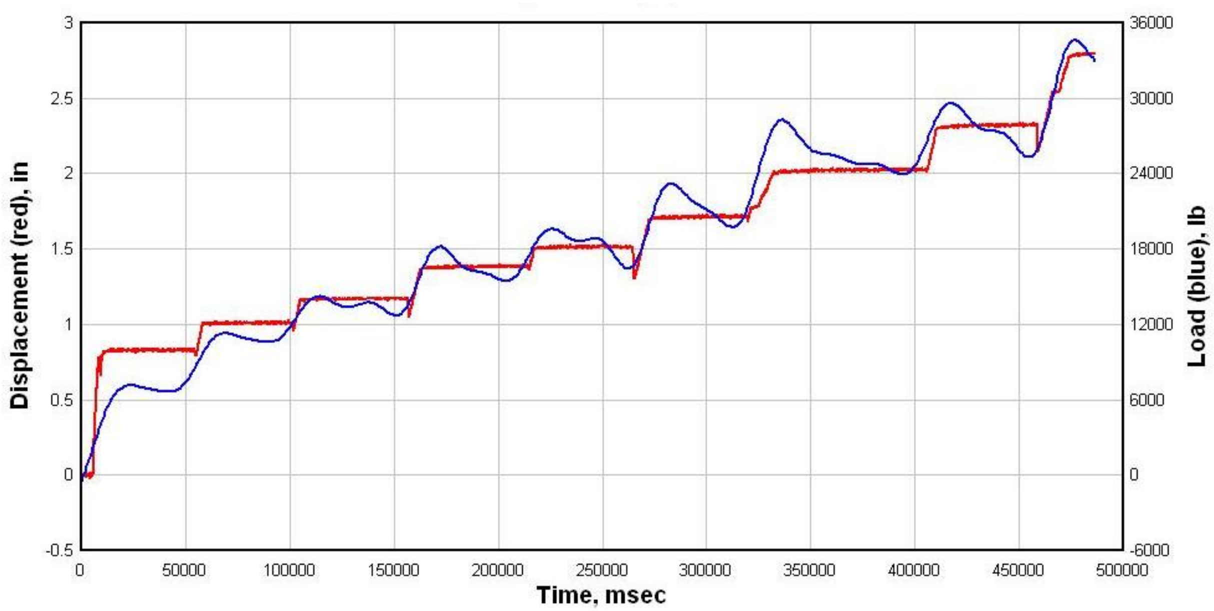

Figure 50. Load-Deflection Plot for Manta Ray Anchor MR-2 at Silver Flag

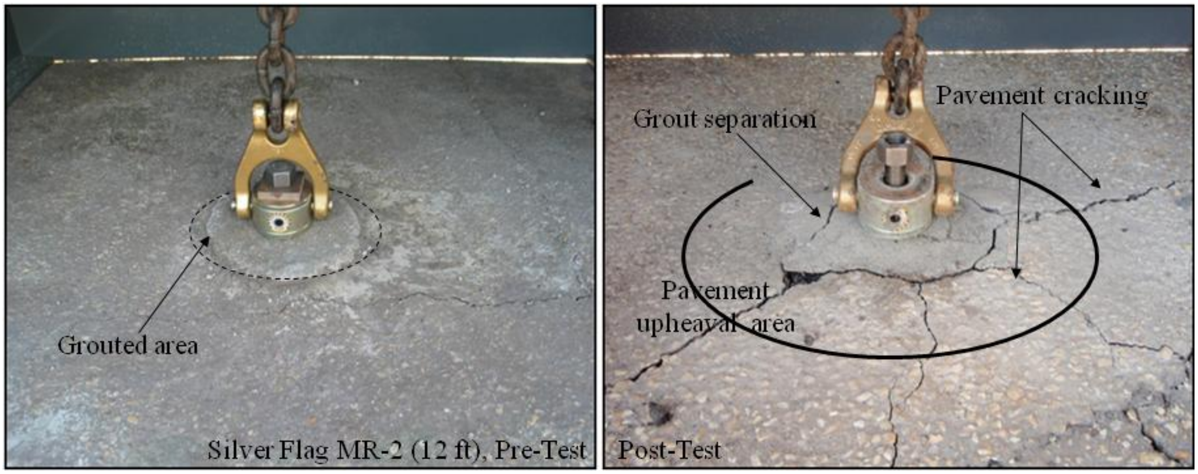

Figure 51. Silver Flag MR-2 (12 ft), Pre-test and Post-Test

Figure 52. Silver Flag MR-3 Anchor Extraction

Several iterations of the testing procedure were necessary (Fig. 53) before the tie-down appeared to lock into place. Uplift capacities ranged from 20,310 lbs to 50,000 lbs. The mean pull-out capacity was 38,410 lbs.

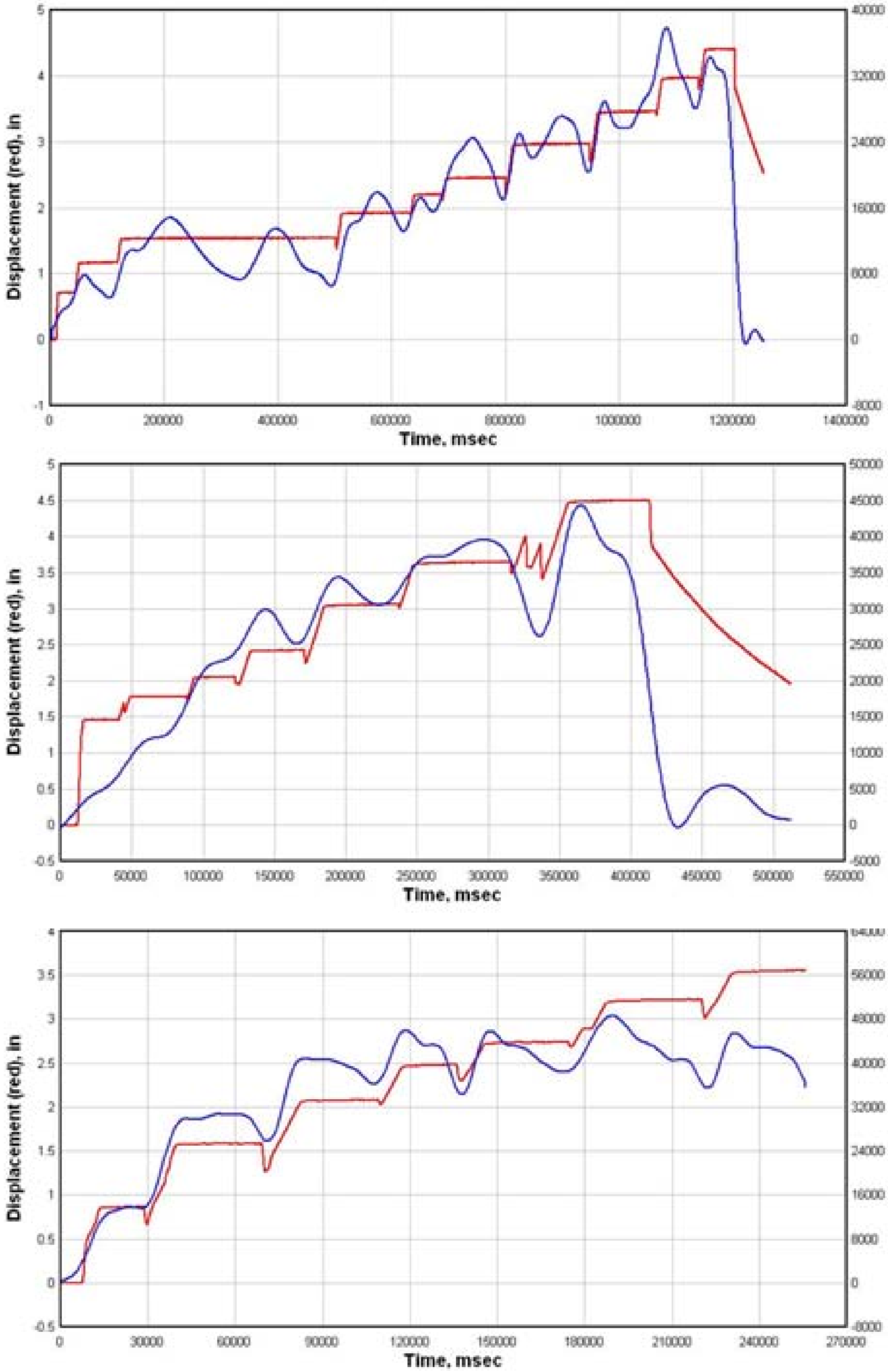

Figure 53. Silver Flag MR-3. Runs 1, 2 and 3

20-ft Installation. Four Manta Ray tie-downs were installed to a depth of 20 ft. MR-1 and MR-2 were subjected to standard loading conditions. Figure 54 shows the load curve generated from load testing of anchor MR-2.

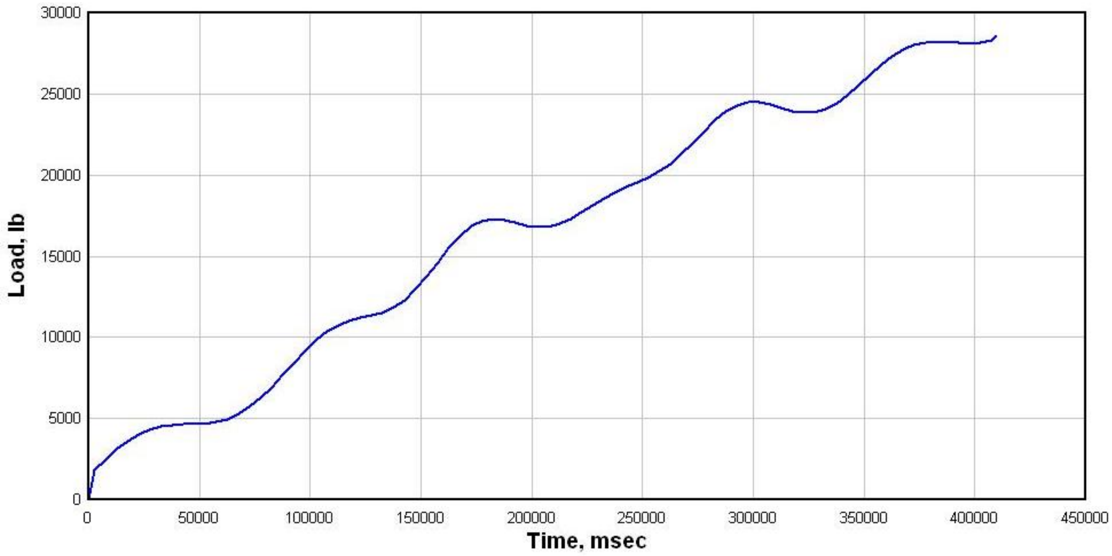

Figure 54. Silver Flag MR-2 (20-ft Installation). No Deflection Data

Tie-downs MR-3 and MR-4 were subjected to slightly different, quasi-dynamic loading conditions. Each of these anchors (MR-3 and MR-4) was loaded to an initial load of 6,500 lbs, for a period of 10 sec, after which the load was removed. This cycle was repeated, with the magnitude of the load increasing in 6,500-lb increments. Uplift capacities ranged from a minimum of 21,890 lbs to a maximum of 37,040 lbs. The mean pull-out capacity was 30,950 lbs.

6.3.3.2. Seguin Auxiliary Airfield

Four Manta Ray earth anchors were installed and load tested at the Seguin Auxiliary Airfield. Each tie-down was driven to a depth of 12 ft, and subsequently subjected to standard loading conditions. Importantly, MR-4 was not grouted prior to load locking. Ultimate uplift capacities of MR-1, MR-2, and MR-3 (the three grouted Manta Rays) ranged from a minimum of 32,160 lbs, to a maximum of 45,570 lbs. The mean pull-out capacity of these three anchors was 40,500 lbs. MR-4, the ungrouted tie-down, reached a pull-out capacity of 29,010 lbs. MR-4 exhibited two more in of deflection than any of the three grouted MR tie-downs tested at Seguin.

6.3.3.3. Avon Park Air Force Range

Four Manta Ray earth anchors were installed and load tested at the Avon Park Range. Three of the anchors were driven to a depth of 12 ft, and one to a depth of 20 ft. MR-2, the 20-ft anchor, was subjected to standard loading conditions. This tie-down achieved and uplift resistance of 31,910 lbs.

Two of the 12-ft anchors, MR-1 and MR-3, were loaded under standard conditions. The remaining 12-ft anchor, MR-4, was loaded to an initial 19,500 lbs for a period of 10 sec. After the initial load was released, MR-4 was reloaded under standard conditions. Uplift capacities ranged from 22,720 lbs to 29,800 lbs. The mean pull-out capacity was 26,160 lbs.

6.3.4. Epoxy Anchoring Systems (AFRL Epoxy, Large Plates, Small Plates)

6.3.4.1. Silver Flag Exercise Site

Four individual AFRL epoxy anchors were installed and load tested at the Silver Flag site. Each anchor was subjected to standard loading conditions. Uplift capacities ranged from 5,350 lbs to 8,750 lbs. The mean pull-out capacity was 7,060 lbs.

Three small plate epoxy anchors were installed and load tested at the Silver Flag site. This anchoring system was comprised of a group of four individual epoxy tie-downs. The plate dimensions were 12 in by 12 in, and the epoxy anchors were spaced 6 in in each direction. Each plate anchor was subjected to standard loading conditions. Uplift capacities ranged from 5,050 lbs to 6,320 lbs. The mean pull-out capacity was 5,780 lbs.

Three large plate epoxy anchors were installed and load tested at the Silver Flag site. This anchoring system comprised a group of four individual epoxy tie-downs. The plate dimensions were 18 in by 18 in, and the epoxy anchors were separated 12 in in each direction. Each plate anchor was subjected to standard loading conditions. Uplift capacities ranged from 8,230 lbs to 11,050 lbs. The mean pull-out capacity was 9,500 lbs. Deflection data were not obtained for this group of mooring points.

6.3.4.2. Seguin Auxiliary Airfield

Three individual AFRL epoxy anchors were installed and load tested at the Seguin Auxiliary Airfield. Each anchor was subjected to standard loading conditions. Uplift capacities ranged from 4,560 lbs to 7,990 lbs. The mean pull-out capacity was 5,930 lbs.

Three small-plate epoxy anchors were installed and load tested at the Seguin Auxiliary Airfield. This anchoring system comprised a group of four individual epoxy tie-downs inserted through full-depth portholes on the plate anchor. The overall plate dimensions measured 12 in by 12 in, and the four individual epoxy anchors were spaced 6 in apart in the *x* and *y* directions. Each plate anchor was subjected to standard loading conditions. Uplift capacities ranged from 4,550 lbs to 6,910 lbs. The mean pull-out capacity was 5,870 lbs.

Three large plate epoxy anchors were installed and load tested at the Seguin Auxiliary Airfield. This anchoring system comprised a group of four individual epoxy tie-downs. The plate dimensions were 18 in by 18 in, and the epoxy anchors were spaced 12 in in the *x* and *y* directions. Each plate anchor was subjected to standard loading conditions. Uplift capacities ranged from 7,910 lbs to 12,070 lbs. The mean pull-out capacity was 10,640 lbs. Figure 55 compares the performance of an individual AFRL epoxy anchor, a small epoxy plate anchor, and a large epoxy plate anchor.

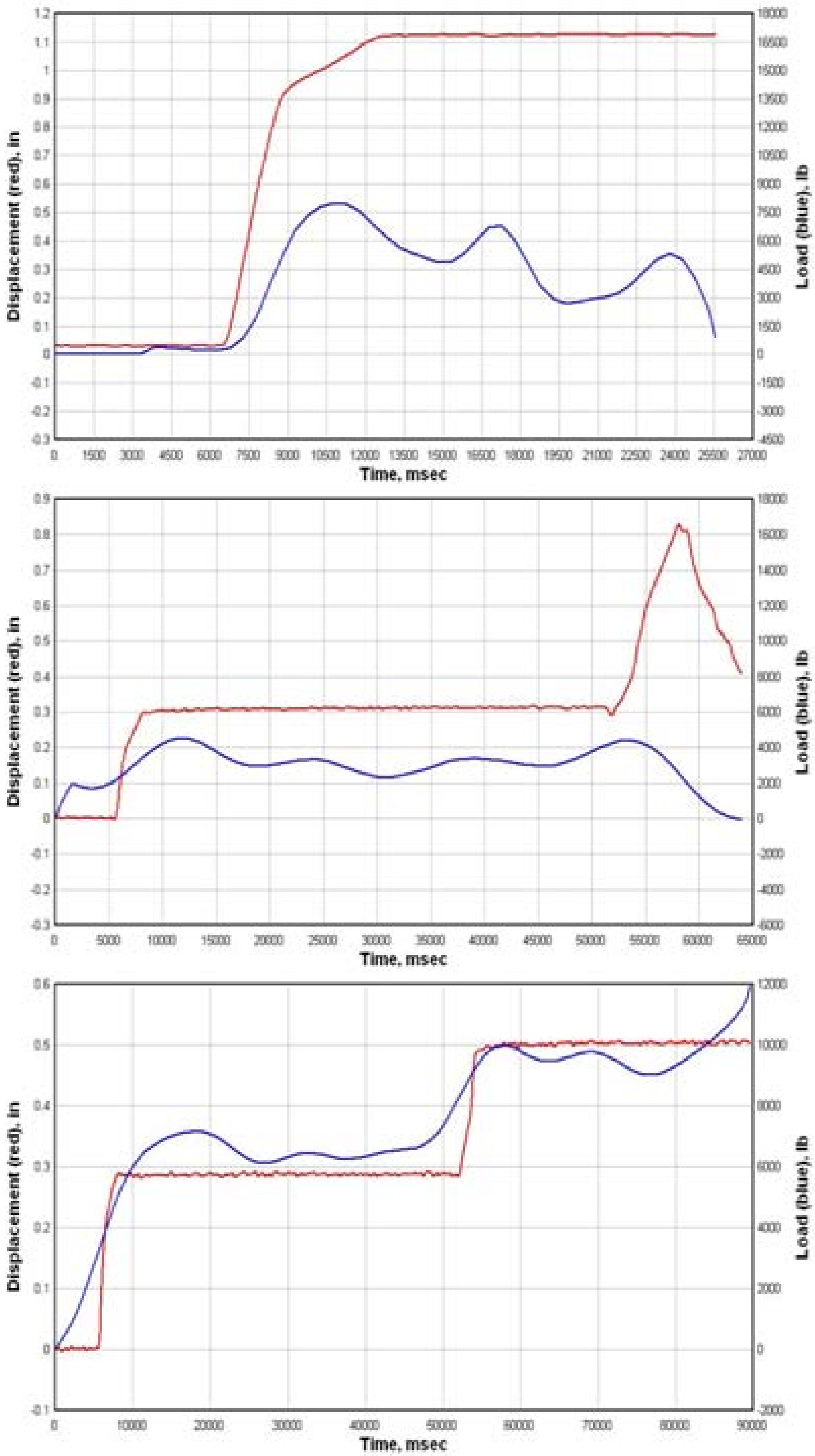

Figure 55. Comparison of Seguin Individual, Small Plate, and Large Plate Epoxy Anchoring Systems

6.3.4.3. Avon Park Air Force Range

Three individual AFRL epoxy anchors were installed and load tested at the Avon Park Range. Each anchor was subjected to standard loading conditions. Uplift capacities ranged from 10,570 lbs to 13,420 lbs. The mean pull-out capacity was 11,780 lbs.

Three small plate epoxy anchors were installed and load tested at the Avon Park Range. This anchoring system was comprised of a group of four individual epoxy tie-downs. The plate dimensions were 12 in by 12 in, and the epoxy anchors were spaced 6 in in each direction. Each plate anchor was subjected to standard loading conditions.

It should be noted that load cell data was not acquired for two of the small plate anchors, SP-2 and SP-3. Estimated peak loads for these two tie-downs were based on pressure gage readings taken during the testing process. This is notated in the data. The estimated pull-out capacities of SP-2 and SP-3 were 16,230 lbs and 17,520 lbs, respectively. Load cell data was available for SP-1. The pull-out capacity of tie-down SP-1 measured 19,190 lbs. Figure 56 compares the performance an individual AFRL epoxy anchor to a small plate epoxy anchor.

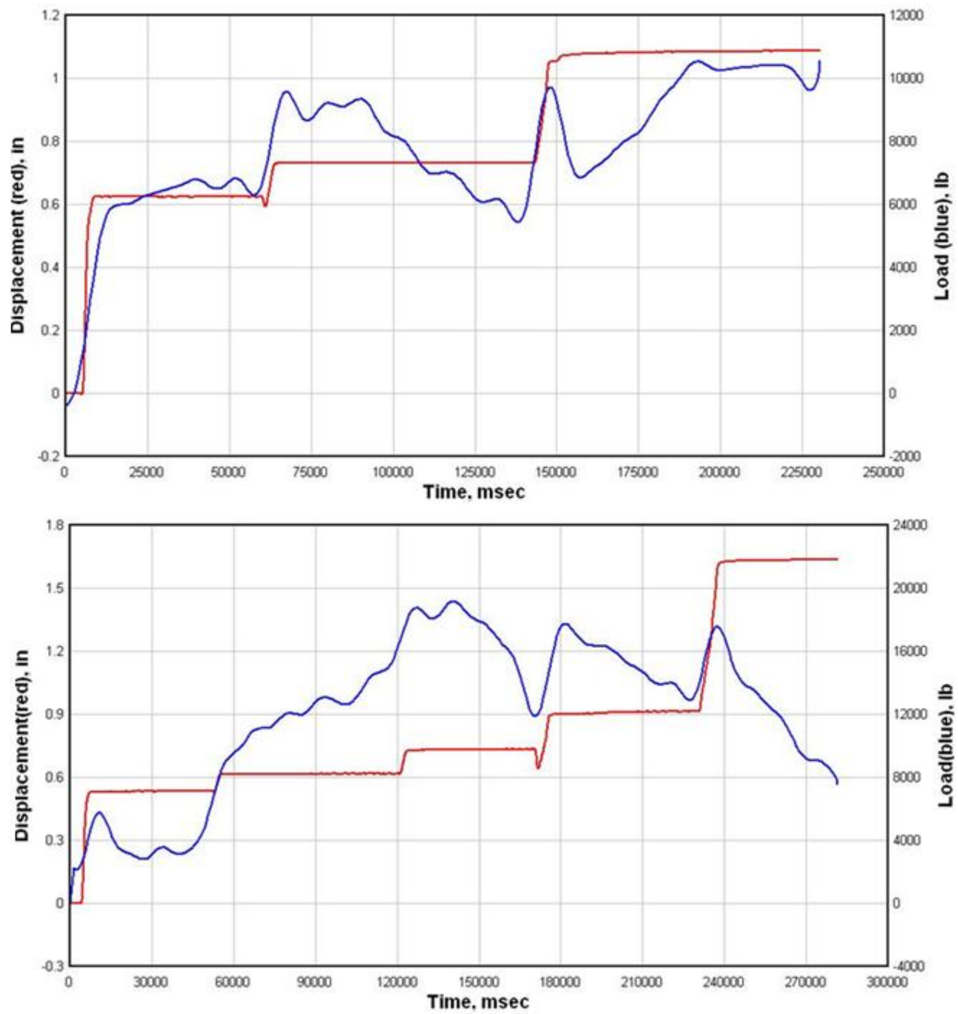

Figure 56. Load and Deflection Data from Individual AFRL Epoxy Anchor and Small Epoxy Plate Anchor

Three large plate epoxy anchors were installed and load tested at the Avon Park Range. This anchoring system was comprised of a group of four individual epoxy tie-downs. The plate dimensions were 18 in by 18 in, and the epoxy anchors were spaced 12 in in each direction. Each plate anchor was subjected to standard loading conditions.

Two of the large plate anchors, LP-1 and LP-2, utilized the bonding agent LiquidRoc 500. The third large plate, LP-3, was bonded with Pavemend, as opposed to LR 500. Load cell data were not available for this group of tie-downs. Peak pull-out capacities were based on pressure gage readings taken during the testing process. This is notated in the data. LP-1 and LP-2 measured estimated peak capacities of 34,400 lbs and 31,150 lbs, respectively. LP-2 post-test pavement damage is shown in Figure 57. LP-3, bonded with Pavemend, measured an estimated peak capacity of 31,150 lbs.

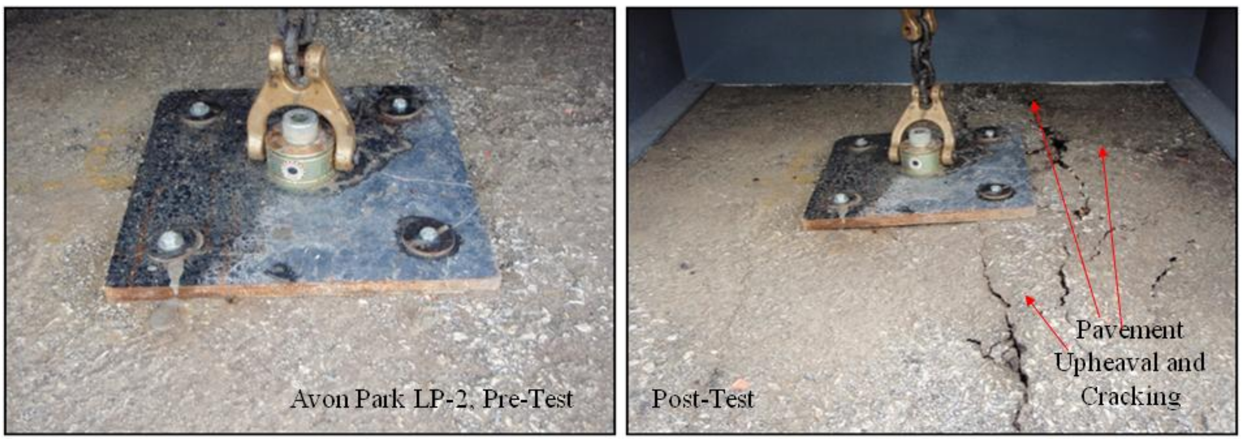

Figure 57. Avon Park LP-2, Post-Test Pavement Damage

6.3.5. Tri-Talon Anchors

6.3.5.1. Silver Flag Exercise Site

Three Tri-Talon tie-downs were installed and load tested at the Silver Flag site. Each of the talon anchors was subjected to standard loading conditions. Uplift capacities ranged from a minimum of 4,340 lbs to a maximum of 6,850 lbs. The mean pull-out capacity measured 5,960 lbs.

6.3.5.2. Avon Park Air Force Range

Five Tri-Talon tie-downs were installed and load tested at the Avon Park Range. Three of the talons were located in Test Area 1, and two of the talons were located in Test Area 2. Each mooring point was subjected to standard loading conditions.

The three talons in Test Area 1 are notated as TT-1, TT-2, and TT-3. Uplift capacities ranged from 21,620 lbs to 24,820 lbs. The mean pull-out capacity measured 23,050 lbs.The two talons in Test Area 2 are labeled as TT-4 and TT-5. Pull-out capacities were 11,140 lbs and 16,410 lbs, respectively. Figure 58 illustrates the load–deflection data collected from TT-2 (Test Area 1) and TT-4 (Test Area 2).

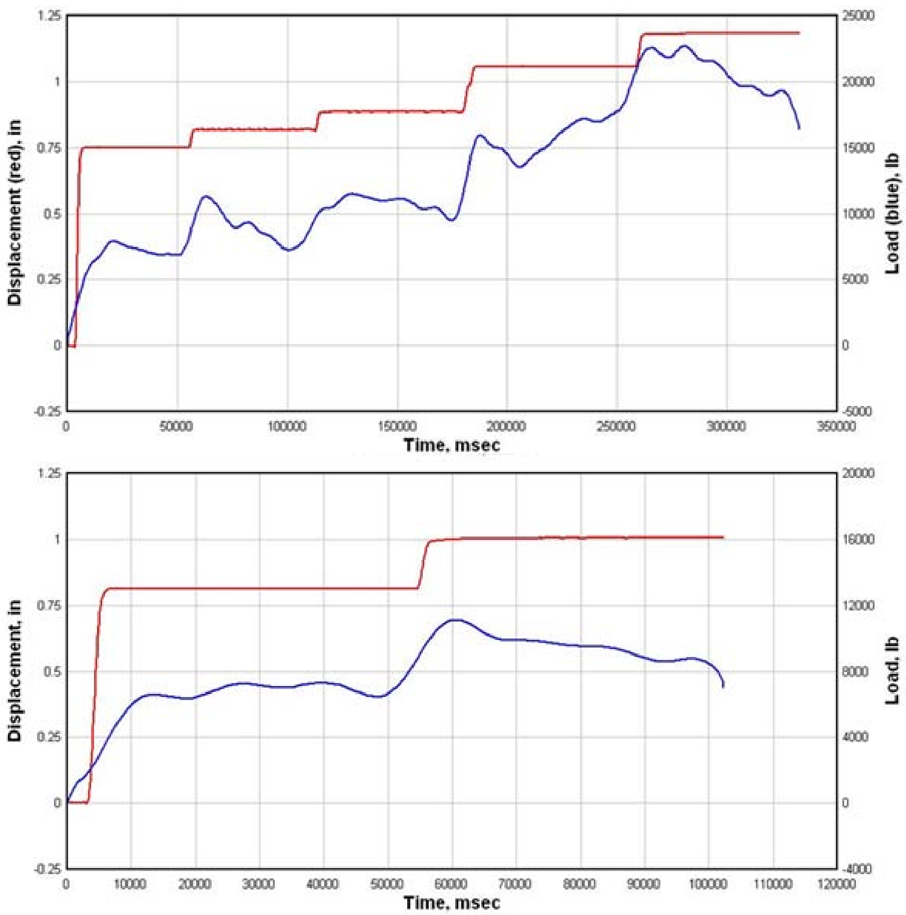

Figure 58. Tri-Talon Anchor Data from Test Areas 1 and 2

6.3.6. AFRL Grouted Anchors

6.3.6.1. Silver Flag Exercise Site

Three AFRL grouted anchors were installed and load tested at the Silver Flag site. AFRL grouted anchors were identical to the AFRL epoxy anchors. However, AFRL grouted anchors utilized a rapid-setting cementitious material as a bonding agent- as opposed to an epoxy mixture.

Load cell data did not transfer for these three tie-downs. Uplift capacities are based on pressure gage readings taken during the testing process. The estimated pull-out resistances ranged from 5,190 lbs to 7,790 lbs. The mean uplift capacity was 6,270 lbs.

6.3.6.2. Avon Park Air Force Range

Three AFRL grouted anchors were installed and load tested at the Avon Park Range. Pull-out capacities ranged from 12,630 lbs to 16,880 lbs. The mean uplift capacity was 14,460 lbs.

6.3.7. Modified AFRL Grouted Anchors

6.3.7.1. Silver Flag Exercise Site

Three modified AFRL grouted anchors were installed and load tested at the Silver Flag site. Uplift capacities ranged from 4,420 lbs to 8,680 lbs. The mean uplift capacity was 6,290 lbs.

Figure 59 compares the performance of an individual AFRL epoxy anchor to that of a Modified AFRL grouted anchor.

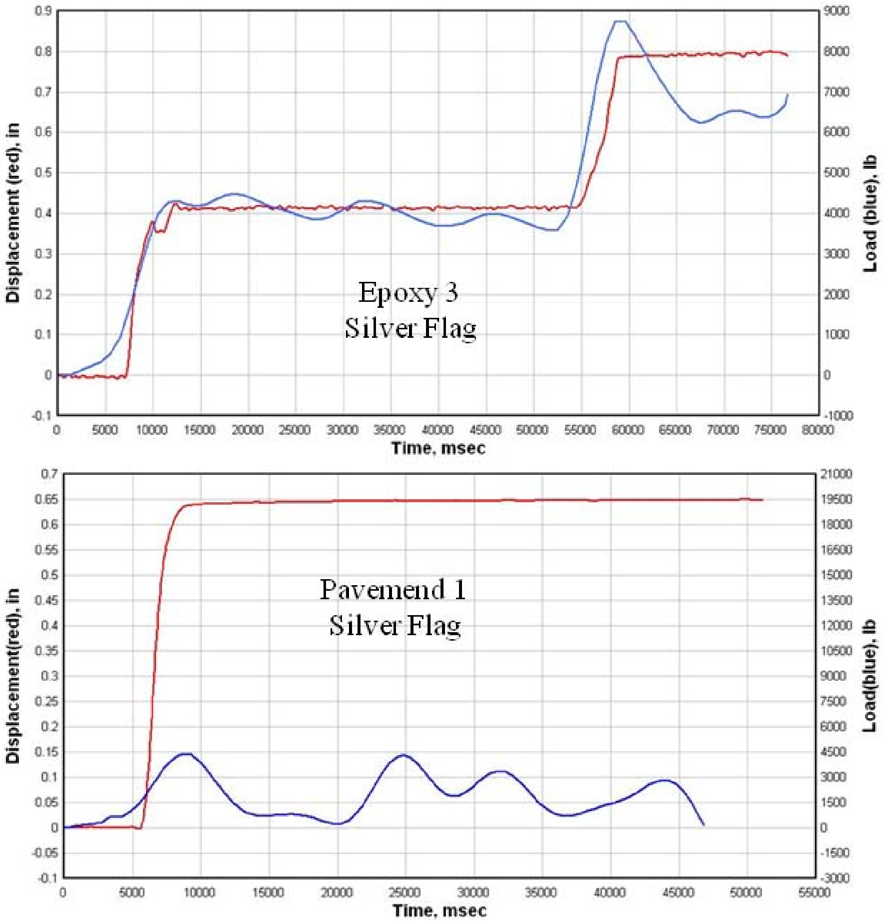

Figure 59. Comparison of Silver Flag AFRL Epoxy and Modified AFRL Grouted AnchorsAvon Park Air Force Range.

Three modified AFRL grouted anchors were installed and load tested at the Silver Flag site. Uplift capacities ranged from 8,850 lbs to 17,310 lbs. The mean uplift capacity was 13,060 lbs. Figure 60 compares a load–deflection data of an individual AFRL epoxy anchor, an AFRL grouted anchor and a Modified AFRL grouted anchor.

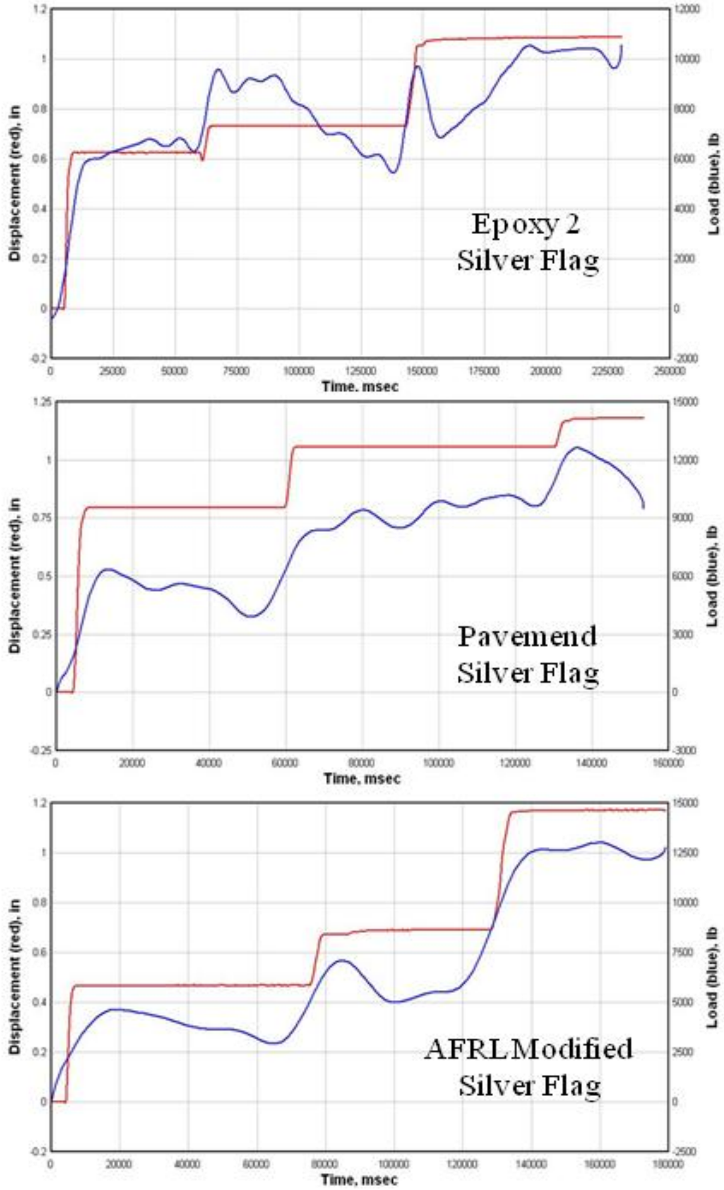

Figure 60. Avon Park- Comparison of Individual AFRL Epoxy Anchor, AFRL Grouted Anchor, and Modified AFRL Grouted Anchor

6.4. Testing Data

The following section includes load and deflection data collected during the testing process. This information includes peak loads estimated from the pressure gage read-outs, which are notated in the data. Tables 7, 8, and 9 detail testing results obtained from the Silver Flag Exercise Site, Seguin Auxiliary Airfield, and Avon Park Air Force Range, respectively. Table 10 illustrates comparisons of anchor performance at each location.

Table 7. Silver Flag Exercise Site Anchor Testing Data

Silver Flag Exercise Site				Test 1		Test 2		Test 3		Test 4		Mean	
Anchor:	Installation Depth (in)	Shaft Diameter (in)	Bonding Agent	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)
Fully Grouted	96	8	Pavemend	9,760	2.16	14,480	2.44	7,350	1.86	12,600	1.69	11,050	2.04
Partially Grouted	96	8	Pavemend	4,830	0.71	10,330	0.73	11,090	2.04	17,020	1.61	10,820	1.27
MR-SR	144		Pavemend	20,310	2.64	34,620	2.8	48,720	12.48[A]	50,000	1.36	38,410	2.27
MR-SR	240		Pavemend	21,890	N/D	28,550	N/D	37,040	N/D	36,320	1.81	30,950	
AFRL Epoxy	18	2	LiquidRoc 500	7,720	0.97	4,340	0.8	6,700	0.76	6,420	0.44	7,060	0.82
Large Plate	18	2	LiquidRoc 500	9,220	N/D	5,190*	N/D	5,840*	N/D	###	###	9,500	N/D
Small Plate	18	2	LiquidRoc 500	5,050	0.65	8,680	0.77	5,770	0.54	###	###	5,780	N/D
Tri-Talon	36	3	Pavemend	6,850	0.8	5,350	1.58	8,750	0.44	###	###	5,960	0.84
AFRL Pavemend	18	2	Pavemend	7,790*	N/D	11,050	N/D	8,230	N/D	###	###	6,270	N/D
Modified AFRL	18	2	Pavemend	4,420	N/D	5,960	N/D	6,320	N/D	###	###	6,290	0.65

Notes:
[A] = Data point not included in mean deflection determination
* = Estimated load based on pressure read-out taken during testing
N/D = No data collected
= Not Tested

Table 8. Seguin Auxiliary Airfield Anchor Testing Data

Seguin Auxiliary Airfield				Test 1		Test 2		Test 3		Mean	
Anchor:	Installation Depth (in)	Shaft Diameter (in)	Bonding Agent	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)
Fully Grouted	96	8	Pavemend	22,510	1.26	29,990	0.75	25,640	0.86	26,050	0.96
Partially Grouted	96	8	Pavemend	14,920	0.57	7,540	0.46	8,170	0.45	10,210	0.49
MR-SR	144		Pavemend	43,780	1.24	45,570	1.38	32,160	1.28	40,500	1.30
AFRL Epoxy	18	2	LiquidRoc 500	4,560	1.66	5,240	1.27	7,990	1.13	5,930	1.35
Large Plate	18	2	LiquidRoc 500	12,070	1.65	11,940	0.51	7,910	0.55	10,640	0.90
Small Plate	18	2	LiquidRoc 500	4,550	0.83	6,140	0.86	6,910	0.5	5,870	0.73
MR-SR (Ungrouted)	144		No Grout	29,010	3.57	###	###	###	###	###	###

Notes:
* = Value based on pressure gage read-outs
= Not tested

Table 9. Avon Park Air Force Range Anchor Testing Data

Avon Park Air Force Range				Test 1		Test 2		Test 3		Mean	
Anchor:	Installation Depth (in)	Shaft Diameter (in)	Bonding Agent	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)	Load (lbs)	Def. (in)
Fully Grouted	96	8	Pavemend	31,750	3.37	23,090	N/D	30,940	1.19	28,590	2.28
Partially Grouted	96	8	Pavemend	13,440	1.14	13,310	0.92	19,080	0.73	15,280	0.93
AFRL Pavemend	18	2	Pavemend	16,880	1.01	13,860	1.32	12,630	1.18	14,460	1.17
Modified AFRL	18	2	Pavemend	13,010	1.17	8,850	1.70	17,310	3.24	13,060	2.04
AFRL Epoxy	18	2	LiquidRoc 500	13,420	0.70	10,570	1.09	11,350	1.26	11,780	1.02
Large Plate	18	2	LiquidRoc 500	34,400	N/D	32,450	N/D	31,150[A,B]	N/D	33,430	N/D
Small Plate	18	2	LiquidRoc 500	19,190	1.64	16,230 *	1.64	17,520*	0.51	17,650	1.26
Tri-Talon TA-1	36	3	Pavemend	24,820	0.92	22,710	1.18	21,620	2.48	23,050	1.53
Tri-Talon TA-2	36	3	Pavemend	11,140	1.00	16,410	1.62	###	###	13,780	1.31
MR-SR	144		Pavemend	29,800	1.01	25,960 *	1.54	22,720*	1.36	26,160	1.30
MR-SR	240		Pavemend	31,910	1.52	###	###	###	###	###	###

Notes:

A= data point not included in mean load or mean deflection determination

B= grouted with Pavemend

*= estimated load based on pressure read-out taken during testing

Table 10. Site Comparison of Anchor Uplift Resistance and Deflection

Location	Silver Flag		Seguin		Avon Park	
Anchor	Load (lbs)	Deflection (in)	Load (lbs)	Deflection (in)	Load (lbs)	Deflection (in)
Fully Grouted Pier	11,050	2.04	26,050	0.96	28,590	2.28
Partially Grouted Pier	10,820	1.27	10,210	0.49	15,280	0.93
MR-SR (12 ft)	38,410	2.27	40,500	1.30	26,160	1.30
AFRL Epoxy	7,270	0.94	5,930	1.35	11,780	1.02
Large Epoxy Plate	9,500	ND	5,870	0.73	33,430	ND
Small Epoxy Plate	5,780	ND	5,870	0.73	17,650	1.26
Tri-Talon	5,960	0.84	###	###	19,340	1.44
AFRL Pavemend	6,270	ND	###	###	13,060	2.04
Modified AFRL Pavemend	6,290	0.65	###	###	13,060	2.04

a. Load (lbs)
b. Deflection (in)
###- Not Tested

7. CONCLUSIONS AND RECOMMENDATIONS

Several of the flexible pavement anchoring systems reached pull-out capacities that could allow them be categorized, based only on loading capacity, as lightweight and/or heavyweight anchors. However, each anchoring system experienced significant permanent deflection. None of the tie-downs tested achieved even the lightweight threshold without displaying plastic displacement. Each anchor is discussed in the following section.

7.1. Fully Grouted Piers

Four 8-in-diameter, 7-ft-deep fully grouted piers were installed at the Silver Flag Exercise Site. These anchors should not be considered for loading applications in silty sands with a relatively high water table. The soil parameters and water intrusion issues at the Silver Flag site did not allow for adequate skin friction to be achieved.

The fully grouted piers installed at Seguin Auxiliary Airfield reached a mean pull-out capacity in excess of 25,000 lbs. However, the mean deflection value was 0.96 in, a substantial portion of which was permanent displacement. There was also noticeable pavement and upheaval. Only in the absence of alternative options should these tie-downs be considered for loading applications in silty clays with CBR values measuring greater than 10.

Three fully grouted piers were installed at the Avon Park Range. The piers reached a mean pull-out capacity in excess of 28,000 lbs. The mean deflection value was 2.28 in. However, FG-3 at Avon Park is probably the best representation of actual aircraft loading demands. This anchor was subjected to an initial quasi-dynamic load demand—19,500 lbs for 10 sec. During the loading stage, FG-3 deflected approximately 0.65 in. After FG-3 was unloaded, the deflection decreased to 0.18 in. As some of the initial 0.65 in of deflection was likely due to seating, FG-3 likely permanently deflected between 0 and 0.18 in as a result of the initial 19,500-lb loading (Fig. 51c). Based on the results from FG-3, fully grouted piers could possibly be recommended for lightweight aircraft tie-downs in flexible pavements underlain by a 6-in-thick cement-stabilized base course and a silty sand sub-grade layer. Further testing, utilizing quasi-dynamic loading demands, is necessary to make this recommendation with a degree of confidence.

7.2. Partially Grouted Piers

Eight-in-diameter, 7-ft-deep partially grouted piers should not be considered for aircraft tie-downs. These anchors failed to reach the lightweight threshold at any of the three locations. Additionally, installation effort was greater for a partially grouted pier than a fully grouted pier.

7.3. Manta Ray Earth Anchors (12-ft Installation Depth)

A total of 16 Manta Ray earth anchors were installed at the three testing sites. Eleven of the Manta Ray tie-downs were installed to a depth of 12 ft, and five Manta Ray tie-downs to a depth of 20 ft. Each of the individual 16 anchors reached the lightweight threshold and four of the anchors reached the heavyweight threshold. Interestingly, the four anchors that achieved the heavyweight loading capacity were all installed at a depth of 12 ft.

Eight Manta Ray earth anchors were installed at the Silver Flag site. Four of the anchors were 12 ft deep, and the remaining four were 20 ft deep. Each anchor was grouted after load-locking. During testing, every anchor experienced significant and unacceptable plastic deformation and pavement damage.

The 12-ft anchors outperformed the 20-ft anchors by nearly 8,000 lbs. This was unexpected. It was expected that driving the anchor deeper into the substrate would enhance performance. Furthermore, a fairly stiff soil was encountered approximately 20 ft below grade. The deeper installation depth and stiff strata failed to improve anchor performance.

Four Manta Ray tie-downs were installed at Seguin Auxiliary Airfield. Three of the anchors were grouted after load-locking and one anchor was left ungrouted. Two of the grouted anchors reached the heavyweight loading threshold. However, each anchor deflected significantly. Additionally, the loading caused substantial pavement upheaval and cracking. The ungrouted anchor deflected more than 3 in, 2 in more than the other three Manta Ray tie-downs installed at Seguin.

Four Manta Ray anchors were installed at the Avon Park Range. Three were 12 ft deep and the fourth was 20 ft deep. All four anchors were grouted after load-locking. Each anchor achieved pull-out capacities in excess of 20,000 lbs. The 20-ft tie-down outperformed the 12-ft tie-downs by roughly 5,000 lbs. However, each of the four anchors deflected significantly.

Based on testing results, Manta Ray earth anchors should not serve as aircraft tie-downs. There was a wide variance in performance and significant deflection.

Also, two problems were noted with the load-locking device. The load-locker failed to lock in at least one of the 16 anchors. This failure surfaced as testing operations commenced and the particular anchor was subjected to a tensile force from the anchor pulling apparatus. The anchor was extracted roughly 2 ft by the anchor puller before appearing to lock into place. It is not uncommon for the load-locker itself to pull a Manta Ray that distance before it locks into place. However, the load-locker had previously operated on this anchor and provided a supposed proof-load and guarantee that the Manta Ray anchor had been rotated into place.

The second issue involving the load-locker was with the theoretical capacity of the hydraulic mechanism itself. The load-locker was designed to proof-load each anchor to 20,000 lbs. However, in every instance, there appeared to be measurable deflection of the anchoring system when a tensile force was applied by the anchor testing apparatus. This deflection started to occur before the anchor puller reached 20,000 lbs, and was evident even with the grouted anchors. This premature deflection did not seem to be an indication that the anchor had receded into the ground after the load locker tension was removed from the anchor system; as the grout was allowed to fully set prior to removing the tensile force being applied by the load locker. After removing the load locker, the height of the extruded rod was measured relative to the pavement surface, and re-measured immediately prior to commencing load testing. There were no instances in which the grouted anchors had measurably receded into the ground in the period between terminating load locking operations and beginning anchor pulling operations.

Another disadvantage of the Manta Ray installation process was the removal of the drive steel. As previously described, drive steel segments are linked together with heavy duty couplers. The drive steel was removed after driving the anchor to the requisite depth. The simplest way to remove the drive steel was to leave the socket adapter connected to the top coupler after terminating the driving process. This allowed for a segment of chain to choke the socket adapter. The chain was simultaneously attached to a fork lift, and the entire length of drive steel was extracted from the ground. That method was very effective for installations 12 ft or less. Deeper installations required much more effort. The drive steel could be extracted only one segment at a time, because the fork lift could not reach high enough to extract the entire drive steel chain. The choke chain had to be re-attached each time a segment was pulled out and removed. When the drive steel chain was reduced to 12 ft it was possible for the fork lift to pull it out in one stroke.

After removing the drive steel, the couplers and extensions had to be detached from one another. This was problematic because soil infiltrated the couplers and stuck to the threads, making it difficult to unscrew the extensions from the couplers. The process was extremely labor intensive.

7.4. Epoxy Anchoring Systems

7.4.1. AFRL Individual Epoxy Anchors

AFRL individual epoxy anchors should not serve as aircraft tie-downs. They failed to reach 17,000 lbs at any of the three locations. The poor pavement conditions contributed to these results, although the cement-stabilized base course at the Avon Park Range improved the tie-down's performance.

7.4.2. Small Epoxy Plate Anchors

AFRL small epoxy plate anchors should not serve as aircraft tie-downs. Interestingly, the small plate group capacity was less than the individual anchor capacity at Silver Flag and Seguin. There was likely a confluence of failure zones that resulted in the decreased capacity.

7.4.3. Large Epoxy Plates

AFRL large epoxy plate anchors were designed to increase individual anchor spacing in comparison to the small plate anchors. The increased spacing improved the anchor performance at each location. Nevertheless, these anchors should not serve as aircraft tie-downs in flexible pavements with a crushed mineral aggregate base course.

Three large plates were installed at Avon Park. Two of the plates utilized an epoxy bonding agent and one plate was bonded with rapid-setting grout. Load capacities for this group of plates were based on pressure gage read-outs recorded during testing. Deflection data was not available. Based on the pressure gage estimates, each of the large plate anchors installed at Avon Park met the lightweight threshold, including the plate bonded with rapid-setting grout. This is important because the epoxy dispensing process required a much greater effort than the grout dispensing process. If each bonding agent delivers roughly the same capacity, the grout is definitely the optimal bonding method. Large epoxy plate and/or rapid-set anchors could possibly serve as lightweight tie-downs in flexible pavements underlain by a cement-stabilized base course and a silty sand sub-grade. Further testing, including quasi-dynamic loading, is necessary before recommending these anchors serve as lightweight tie-downs.

7.5. Tri-Talon Anchors

As expected, Tri-Talon anchors installed at Avon Park significantly outperformed Tri-Talon anchors installed at Silver Flag. This was due to the more competent soil matrix and cement-stabilized base course at Avon Park. The three talons installed in Test Area 1 at Avon Park each achieved pull-out resistance capacities in excess of 20,000 lbs, although there was substantial anchor displacement and pavement deformation from each of the talons.

Based on testing, tri-talon tie-downs should not serve as aircraft mooring points, except possibly in flexible pavements with a cement-stabilized base course. Further testing, focusing on quasi-dynamic loading conditions, is necessary before recommending this option.

7.6. AFRL Grouted Anchors and Modified AFRL Grouted Anchors

Based on testing, these two anchor types should not serve as aircraft tie-downs. They represented a more attractive option than the epoxy anchors because of the reduced installation effort, but failed to reach the lightweight capacity threshold.

7.7. Recommendations

AFRL recommends additional testing focusing on the following loading conditions:

1) three cycles of quasi-dynamic loading, 17,000 lbs applied for 10 sec;
2) three cycles of quasi-dynamic loading, 37,700 lbs applied for 10 sec; and
3) a 19.15° pull angle relative to the pavement surface.

AFRL recommends additional testing of the following anchoring systems to determine if they are capable of meeting lightweight and heavyweight loading capacity:

1) fully grouted piers;
2) AFRL grouted anchors, spaced more than 12 in;
3) Sting Ray earth anchors;
4) Tri-Talon anchors;
5) helical anchors.

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Appendix A: Eglin AFB, FL, Rigid Pavement Tie-down Data

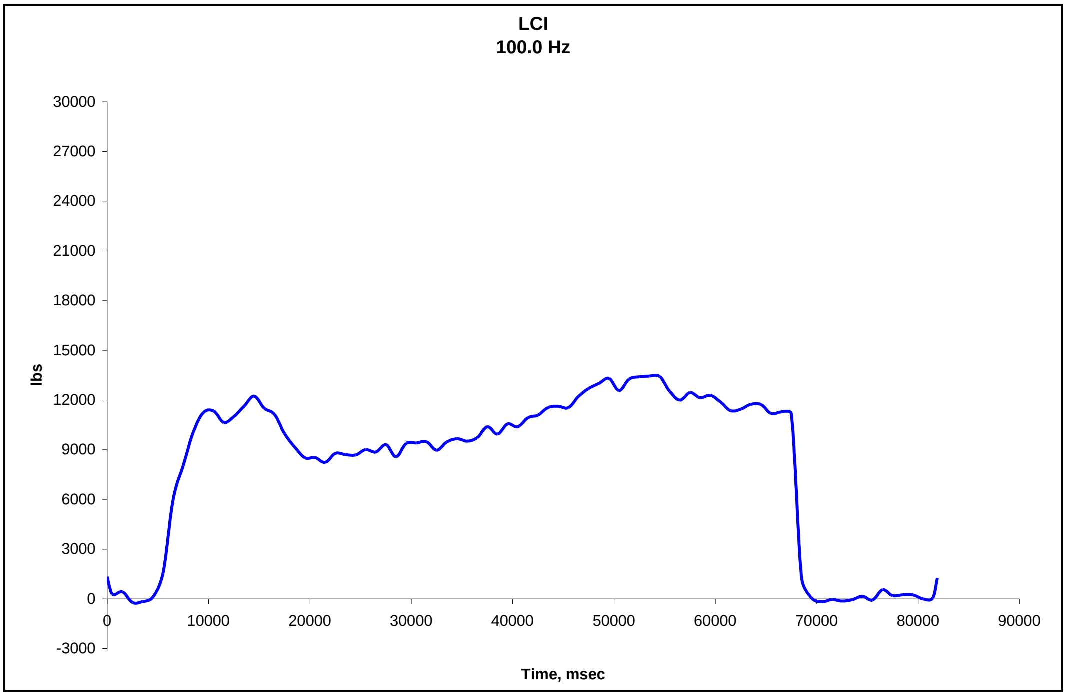


Figure A-1. Tie-down 1: Load Cell Data

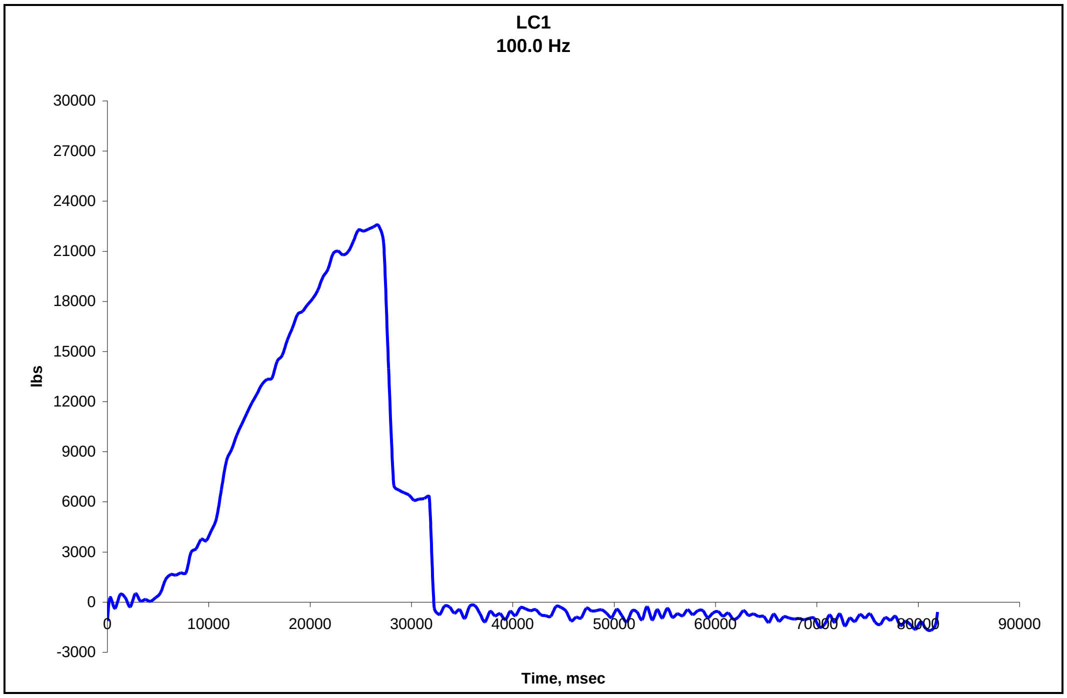


Figure A-2. Tie-down 2: Load Cell Data

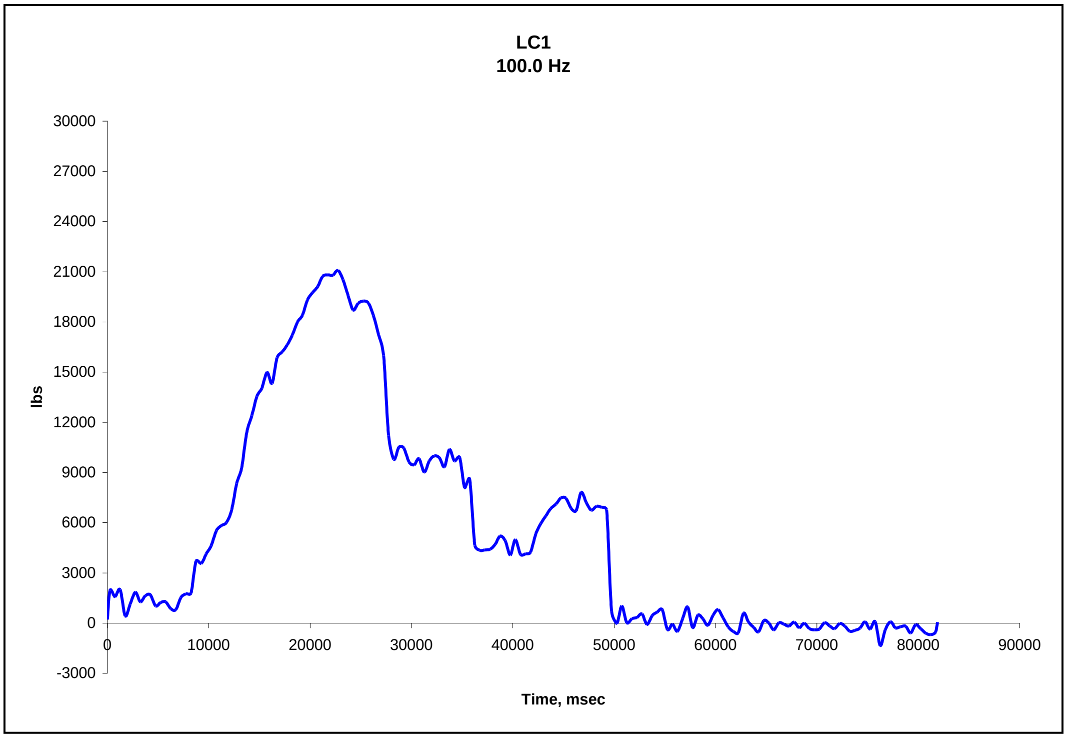


Figure A-3. Tie-down 3: Load Cell Data

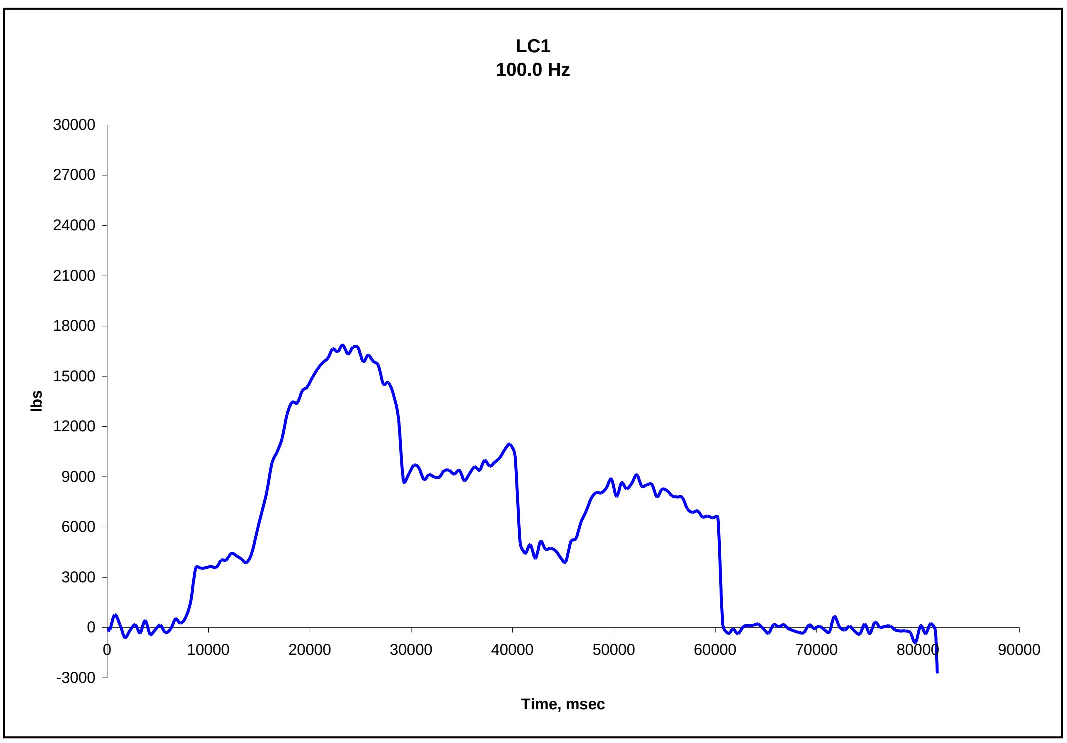


Figure A-4. Tie-down 4: Load Cell Data

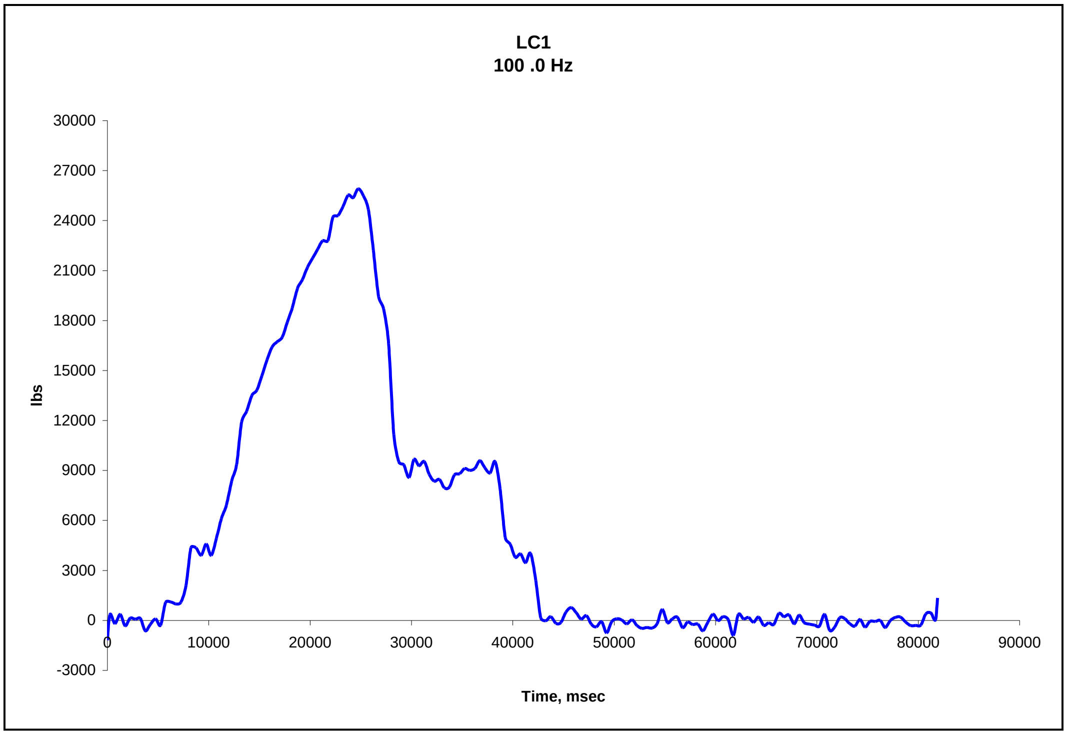


Figure A-5. Tie-down 5: Load Cell Data

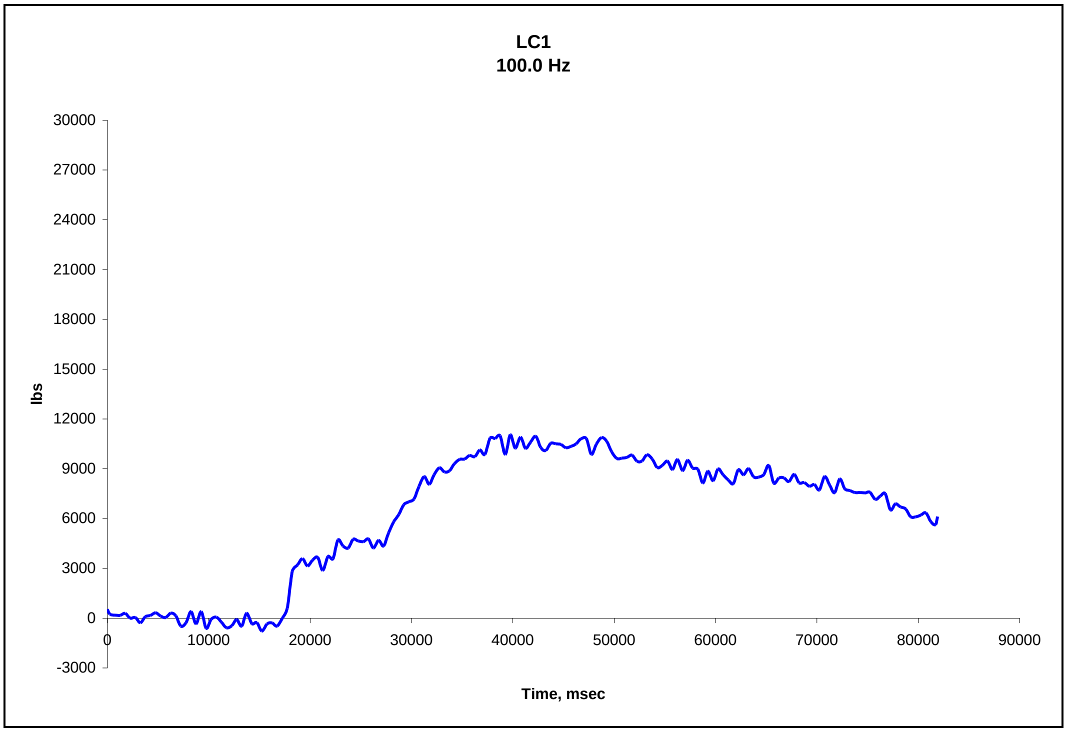


Figure A-6. Tie-down 6: Load Cell Data

Appendix B: Installation Timelines and Equipment Lists for Rigid Pavement Tie-downs in Contingency Environments.

Table B-1. Neenah Mooring Eye without Concrete Pier

<table>
<tr><th colspan="2">Timeline</th><th>Requisite Equipment</th></tr>
<tr><th>Task</th><th>Time (mins)</th><td rowspan="9">Neenah mooring eye
#3 (3/8-in-diameter) rebar
Rebar cutting tool
Hilti drill (or equivalent) with 1¼-in-diameter drill bit
Duct tape
Husqvarna Model 6600 (66 HP) walk-behind concrete saw (or equivalent)
Skid steer (or equivalent) with impactor and bucket attachments, or 90-lb Jackhammer (with 110-psi air compressor)
Plate compactor
Transit concrete mixer
Concrete vibrator
Hand tools (for concrete finishing)</td></tr>
<tr><td>Saw-cutting</td><td>45</td></tr>
<tr><td>Impacting and Debris Removal</td><td>45</td></tr>
<tr><td>Base Prep</td><td>30</td></tr>
<tr><td>Drilling</td><td>45</td></tr>
<tr><td>Mooring Eye Placement</td><td>15</td></tr>
<tr><td>PCC Placement</td><td>30</td></tr>
<tr><td>Total</td><td>210</td></tr>
<tr><td colspan="2"></td></tr>
</table>

Table B- 2. Neenah Mooring Eye with Concrete Pier

<table>
<tr><th colspan="2">Timeline</th><th>Requisite Equipment</th></tr>
<tr><th>Task</th><th>Time (mins)</th><td rowspan="10">Neenah mooring eye
#3 (3/8-in diameter) rebar
Rebar cutting tool
Duct tape
Husqvarna Model 6600 (66-hp) walk-behind concrete saw (or equivalent)
Skid steer (or equivalent) with impactor and bucket attachments, or 90-lb Jackhammer (with 110-psi air compressor)
Transit concrete mixer
Concrete vibrator
Hand tools (for concrete finishing)
#4 (1/2-in-diameter) rebar (various pieces)
Rebar ties
Line truck with 24-in auger capability</td></tr>
<tr><td>Saw-cutting</td><td>45</td></tr>
<tr><td>Impacting and Debris Removal</td><td>45</td></tr>
<tr><td>Hole Auger</td><td>30</td></tr>
<tr><td>Rebar Cage Construction</td><td>360</td></tr>
<tr><td>Sleeve and Pier Insertion</td><td>30</td></tr>
<tr><td>Mooring Eye Placement</td><td>15</td></tr>
<tr><td>PCC Placement</td><td>30</td></tr>
<tr><td>Total</td><td>555</td></tr>
<tr><td colspan="2"></td></tr>
</table>

Table B-3, Hat-Type Mooring Point Saw-cut Method

<table>
<tr><th colspan="2">Timeline</th><th>Requisite Equipment</th></tr>
<tr><th>Task</th><th>Time (mins)</th><td rowspan="8">• Hat-Type mooring point
• Husqvarna Model 6600 (66-hp) walk-behind concrete saw (or equivalent)
• Skid steer (or equivalent) with impactor and bucket attachments, or 90-lb Jackhammer (with 110-psi air compressor)
• Lumber (for suspending hat tie during concrete placement)
• Transmit concrete mixer
• Concrete vibrator
• Hand tools (for finishing concrete)
• Water distributor and pump
• Shop vacuum</td></tr>
<tr><td>Saw-cutting</td><td>45</td></tr>
<tr><td>Impacting and Debris Removal</td><td>45</td></tr>
<tr><td>Base Course Excavation</td><td>60</td></tr>
<tr><td>Anchor Placement</td><td>15</td></tr>
<tr><td>PCC Placement</td><td>30</td></tr>
<tr><td>Total</td><td>195</td></tr>
<tr><td colspan="2"></td></tr>
</table>

Table B-4, Hat-Type Mooring Point Coring Method

<table>
<tr><th colspan="2">Timeline</th><th>Requisite Equipment</th></tr>
<tr><th>Task</th><th>Time (mins)</th><td rowspan="7">• Hat-Type mooring point
• Coring rig
• Lumber (for suspending hat tie during concrete placement)
• Transmit concrete mixer
• Concrete vibrator
• Hand tools (for finishing concrete)
• Water distributor and pump
• Shop vacuum</td></tr>
<tr><td>Coring</td><td>60</td></tr>
<tr><td>Core Extraction</td><td>5</td></tr>
<tr><td>Base Course Excavation</td><td>60</td></tr>
<tr><td>Anchor Placement</td><td>15</td></tr>
<tr><td>PCC Placement</td><td>30</td></tr>
<tr><td>Total</td><td>170</td></tr>
</table>

Appendix C: Installation Timelines and Equipment Lists for Flexible Pavement Anchoring Systems

Table C-1. Fully Grouted Piers

<table>
<tr><th colspan="2">Timeline</th><th>Requisite Equipment</th></tr>
<tr><th>Task</th><th>Time (mins)</th><td rowspan="8">
1-in-diameter threaded rod (10 ft long) with hardware (lifting hoist ring, nuts, washers)
6-in diameter by 12-in length cylinder mold (for concrete plug)
Pavemend 15.0 (8–10 5-gal buckets)
Water source and liquid measuring cup
Electric handheld concrete mixer with compatible paddle
Shovels (square and round point), brooms, and wrenches (gear and pipe)
Electric hand held concrete vibrator
Skid steer w/auger attachment and 4-ft x 8-in-diameter bore bit with extensions
Coring rig with 8-in-diameter core bit
Portable band saw and grinder (for trimming threaded rod)
Plastic wrap or duct tape (protects protruding threads during grout placement)
Portable generator and extension cords
</td></tr>
<tr><td>Constructing Plug</td><td>10</td></tr>
<tr><td>Coring Operations</td><td>5</td></tr>
<tr><td>Augering Hole</td><td>20</td></tr>
<tr><td>Tendon Insertion and Grout Placement</td><td>30</td></tr>
<tr><td>Tendon Trimming</td><td>15</td></tr>
<tr><td>Total</td><td>80</td></tr>
<tr><td colspan="2"></td></tr>
</table>

Table C-2. Partially Grouted Piers

<table>
<tr><th colspan="2">Timeline</th><th>Requisite Equipment</th></tr>
<tr><th>Task</th><th>Time (mins)</th><td rowspan="10">
1-in-diameter threaded rod (10 ft long) with hardware (lifting hoist ring, nuts, washers)
6-in by 12-in cylinder mold (for concrete plug)
Pavemend 15.0 (three 5-gal buckets)
Water source and liquid measuring cup
Electric hand-held concrete mixer with compatible paddle
Shovels (square and round point), rakes, weighted hand-held tamping rods, brooms, and wrenches (gear and pipe)
Electric hand held concrete vibrator
Skid steer w/auger attachment and 4-ft x 8-in-diameter bore bit with extensions
Coring rig with 8-in-diameter core bit
Portable band-saw and grinder (for trimming threaded rod)
Plastic wrap or duct tape (protects protruding threads during grout placement)
Portable generator and extension cords
Chemical stabilizing material (Portland)
</td></tr>
<tr><td>Constructing Plug</td><td>10</td></tr>
<tr><td>Coring Operations</td><td>5</td></tr>
<tr><td>Augering Hole</td><td>20</td></tr>
<tr><td>Chemically Stabilizing Removed Loose Soil</td><td>10</td></tr>
<tr><td>Tendon Insertion and Grout Placement</td><td>5</td></tr>
<tr><td>Hand Compacting Stabilized Backfill</td><td>15</td></tr>
<tr><td>Grout: Mixing, Placement and Vibrateion (for 12-in Surface Cap)</td><td>5</td></tr>
<tr><td>Tendon Trimming</td><td>15</td></tr>
<tr><td>Total</td><td>85</td></tr>
</table>

Table C-3. Manta Ray Earth Anchors

Timeline		Requisite Equipment
Task	**Time (min)**	• Five 1-in-diameter threaded rods (3-ft lengths) with hardware (lifting hoist ring, nuts, washers, couplers) • Complete Manta Ray anchor system to include five additional 28-in extensions • Pavemend 15.0 (three 5-gal buckets) • Water source and liquid measuring cup • Electric hand held concrete mixer with compatible paddle • Shovels (square and round point), brooms, and wrenches (gear and pipe) • Electric hand-held concrete vibrator • Skid steer w/impactor attachment containing blunt tip concrete breaker • Coring rig with 12-in-diameter core bit • Portable band-saw and grinder (for trimming threaded rod) • Plastic wrap or duct tape (protects protruding threads during grout placement) • Portable generator and extension cords
Coring Operations	5	
Anchor Installation	20	
Pre-tensioning Anchor (Hydraulic Load Locker System)	20	
Grout: Mixing, Placement and Vibrateion (for 12-in Surface Cap)	5	
Drive Steel Removal	20*	
Tendon Trimming	15	
Total	85	

* This time can be substantially longer if Manta Ray install depth is greater than 15 ft

Table C-4. Tri-Talon Anchors

Timeline		Requisite Equipment
Task	**Time (min)**	• One Tri-Talon Anchor system with manual drive rod and associated hardware (lifting hoist ring, nuts, bolts, washers) • Electric hand-held concrete drill with 2-in drill bit (24-in length) • Portable shop vacuum with attachments • Pavemend 15.0 (one 1-gal bucket) • Water source and liquid measuring cup • Electric hand-held concrete mixer with compatible paddle • Shovels (square and round point) • Electric impact with deep well sockets and standard mechanics wrenches • Coring rig with 2-in-diameter core bit (high-quality stabilized base-course areas only) • Plastic wrap or duct tape (protects threaded coupler during grout placement) • Portable generator and extension cords
Pavement Coring (Needed Only for Pavements Containing High-quality Stabilized Base Course Layer	5	
Hand Drilling/Augering and Vacuuming Hole	10	
Tendon Insertion and Grout Placement	5	
Total	**20**	

Table C-5. AFRL Epoxy Anchors

Timeline		Requisite Equipment
Task	**Time (min)**	• One AFRL Epoxy Anchor system with associated hardware (lifting hoist ring, nuts, bolts, washers) • Electric hand-held concrete drill with 2-in drill bit (18-in length) • Portable shop vacuum with attachments • Liquidroc 500 (seven 8.5-ounce caulk tubes) • 18-volt battery- or electric-powered caulk gun • Electric impact wrench with deep well sockets; standard mechanic's wrenches • Coring rig with 2-in-diameter core bit (high-quality stabilized base course areas only) • Portable generator and extension cords
Pavement Coring (Needed Only for Pavements Containing High-quality Stabilized Base Course Layer	5	
Hand Drilling/Augering and Vacuuming Hole	10	
Epoxy Mixing and Placement	10	
Anchor Insertion	1	
Total	**26**	

Table C-6. Epoxy Plate Anchors

Timeline		Requisite Equipment
Task	**Time (min)**	• One epoxy anchor system with associated hardware (lifting hoist ring, nuts, bolts, washers) • Electric hand-held concrete drill with 2-in drill bit (18-in length) • Portable shop vacuum with attachments • Liquidroc 500 (28 8.5-ounce caulk tubes) • Two 18-volt battery- or electric-powered caulk guns • Electric impact wrench with deep well sockets; standard mechanic's wrenches • Coring rig with 2-in-diameter core bit (high-quality stabilized base course areas only) • Portable generator and extension cords
Pavement Coring (Needed Only for Pavements Containing High-quality Stabilized Base Course Layer	15	
Hand Drilling/Augering and Vacuuming Hole	25	
Fastening Plate to Hoist Ring and Placement	3	
Epoxy Mixing and Placement	30	
Anchor Insertion	2	
Total	**75**	

Table C-7. AFRL Pavemend-15 Anchors

Timeline		Requisite Equipment
Task	**Time (mins)**	• One AFRL Pavemend-15 Anchor system with associated hardware (lifting hoist ring, nuts, bolts, washers) • Electric hand-held concrete drill with 2-in drill bit (18-in length) • Portable shop vacuum with attachments • Pavemend-15 (1-gal bucket) • Water source and measuring cup • Electric impact wrench with deep well sockets; standard mechanic's wrenches • Coring rig with 2-in-diameter core bit (high-quality stabilized base course areas only) • Electric hand-held concrete mixer with compatible paddle • Portable generator and extension cords
Pavement Coring (Needed Only for Pavements Containing High-quality Stabilized Base Course Layer	5	
Hand Drilling/Augering and Vacuuming Hole	10	
Grout Mixing and Placement	5	
Anchor Insertion	1	
Total	**26**	

Table C-8. Modified AFRL Pavemend-15 Anchors

Timeline		Requisite Equipment
Task	**Time (mins)**	• One Modified AFRL Pavemend-15 Anchor system with associated hardware (lifting hoist ring, nuts, bolts, washers) • Electric hand-held concrete drill with 2-in drill bit (18-in length) • Portable shop vacuum with attachments • Pavemend-15 (1-gal bucket) • Water source and measuring cup • Electric impact wrench with deep well sockets; standard mechanic's wrenches • Coring rig with 2-in-diameter core bit (high-quality stabilized base course areas only) • Electric hand-held concrete mixer with compatible paddle • Portable generator and extension cords
Pavement Coring (Needed Only for Pavements Containing High-quality Stabilized Base Course Layer	5	
Hand Drilling/Augering and Vacuuming Hole	10	
Grout Mixing and Placement	5	
Anchor Insertion	1	
Total	**26**	

Appendix D: Predicted vs. Measured Fully Grouted Pier Uplift Capacity

Table D-1 provides empirical skin friction values derived from extensive field testing conducted on concrete piles. Table D-2 illustrates a comparison of predicted fully grouted pier pull-out resistance versus experimentally determined pull-out resistance. Theoretical uplift resistance values were computed with the following formula:

$$\boldsymbol{P = w + P_{\mathrm{fr}}}$$

where
P = Pullout force needed for extraction
w = Weight of the pier
P_{fr} = Skin friction of the pier (Table D-1)

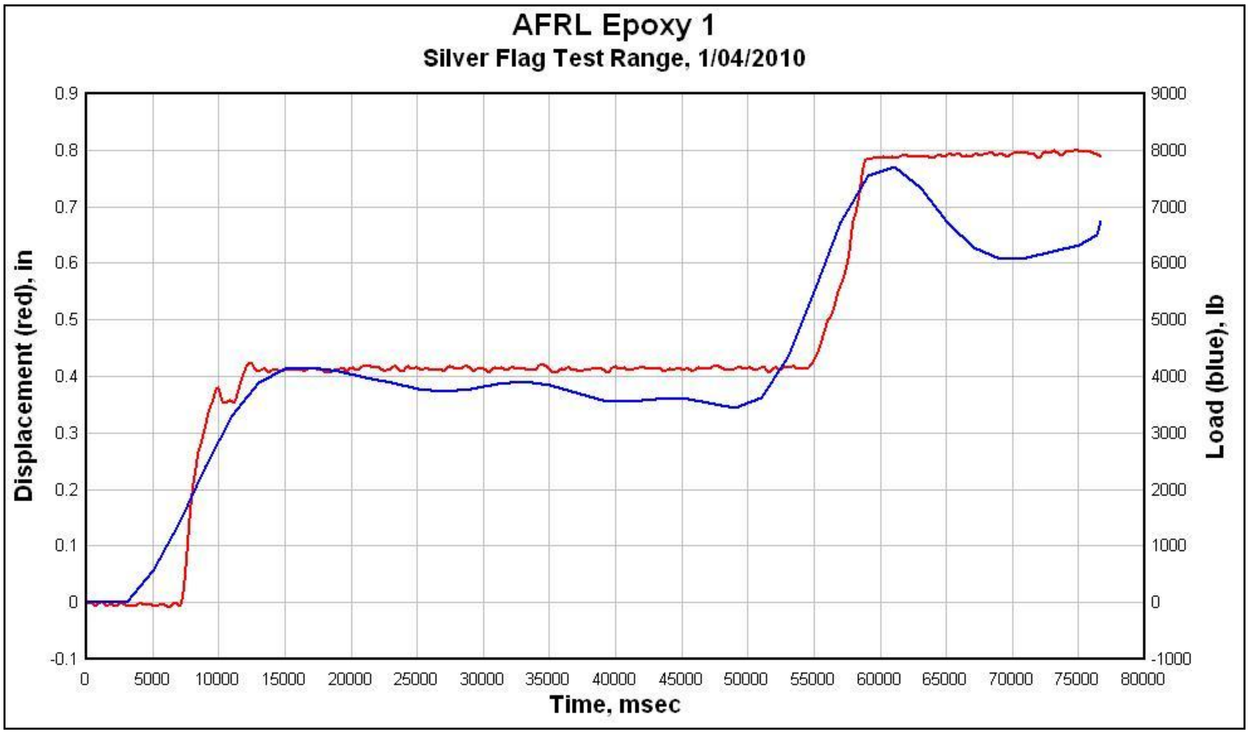
The skin friction, *s*, can be calculated for a pier by selecting the soil type in Table 5 and multiplying it by its surface area(13). Silver Flag soil conditions are best represented by the lower bounds of the silty sand range. Seguin soil conditions are best described as firm clay, with skin friction values likely at the upper bound of the firm clay range. Avon Park site conditions are best represented by the lower bounds of the silty sand range.

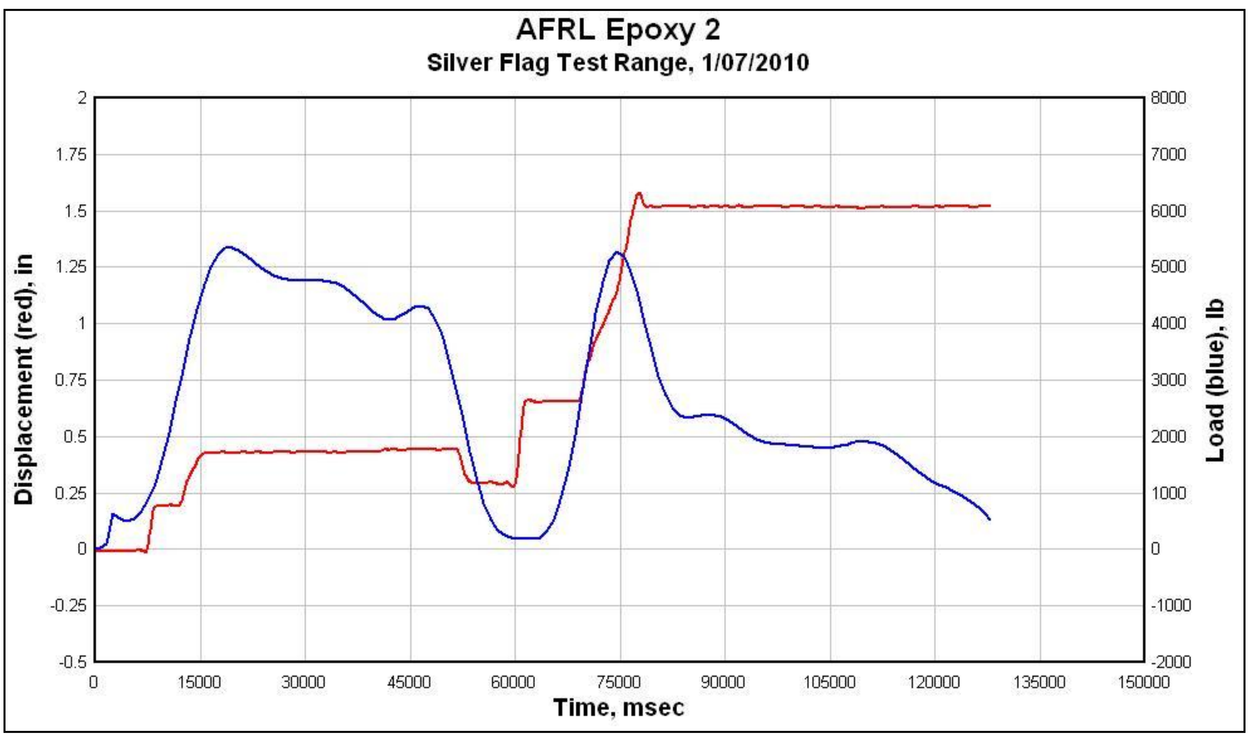
Table D-1 compares the predicted fully grouted pier capacity to the measured pull-out capacity. The predictive model provides a reasonable approximation for the Silver flag location. However, the model is fairly conservative in regards to Seguin and Avon Park. This is possibly due to the fact that the model does not take into account the adhesive bond between the pile and the base course/pavement matrix.

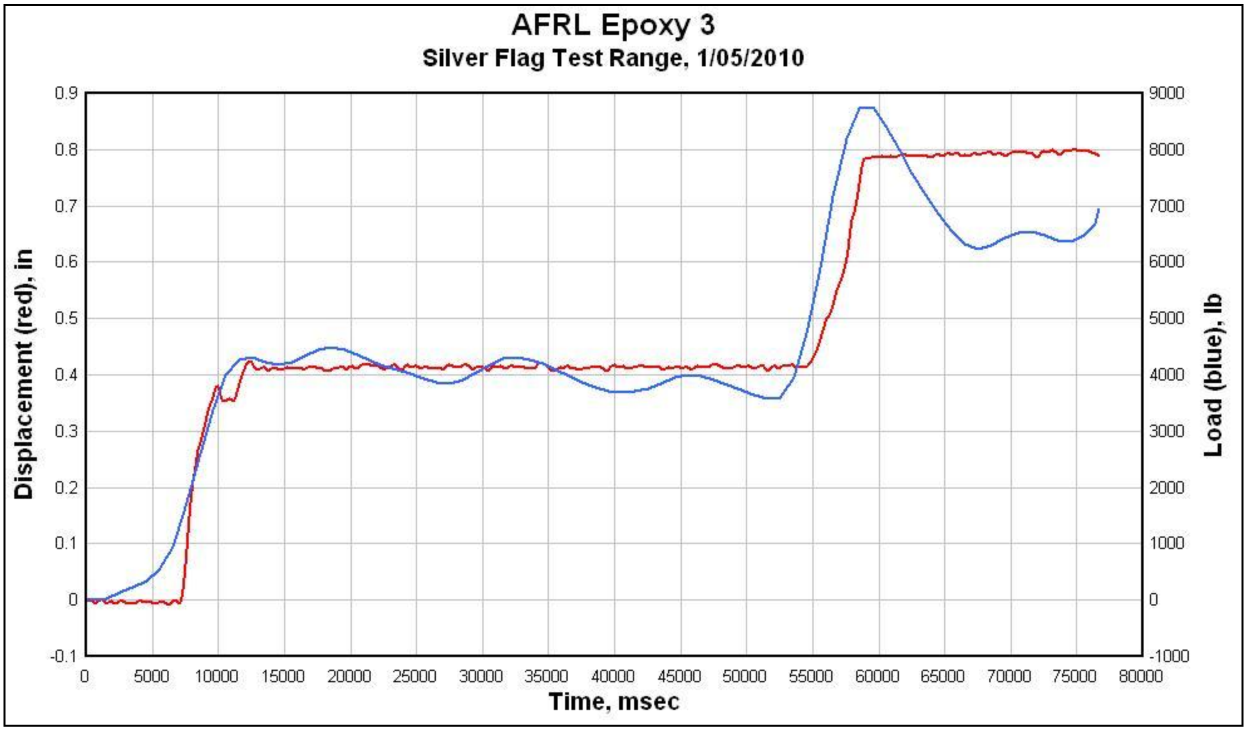
Table D-1. Comparison of Predicted and Measured Fully Grouted Pier Capacities for a 330-lb Pile and 14.67-ft^2 Surface Area

Location	Soil Type	Predicted Skin Friction (lbs/ft^2)		Pull-out Resistance (lbs)	
		Lower	Upper	Predicted	Measured
Silver Flag	Silty Sand	8800	14670	9130–15000	11050
Seguin	Firm Clay	10270	16140	10100–16740	26050
Avon Park	Silty Sand	8800	14670	9130–15000	28590

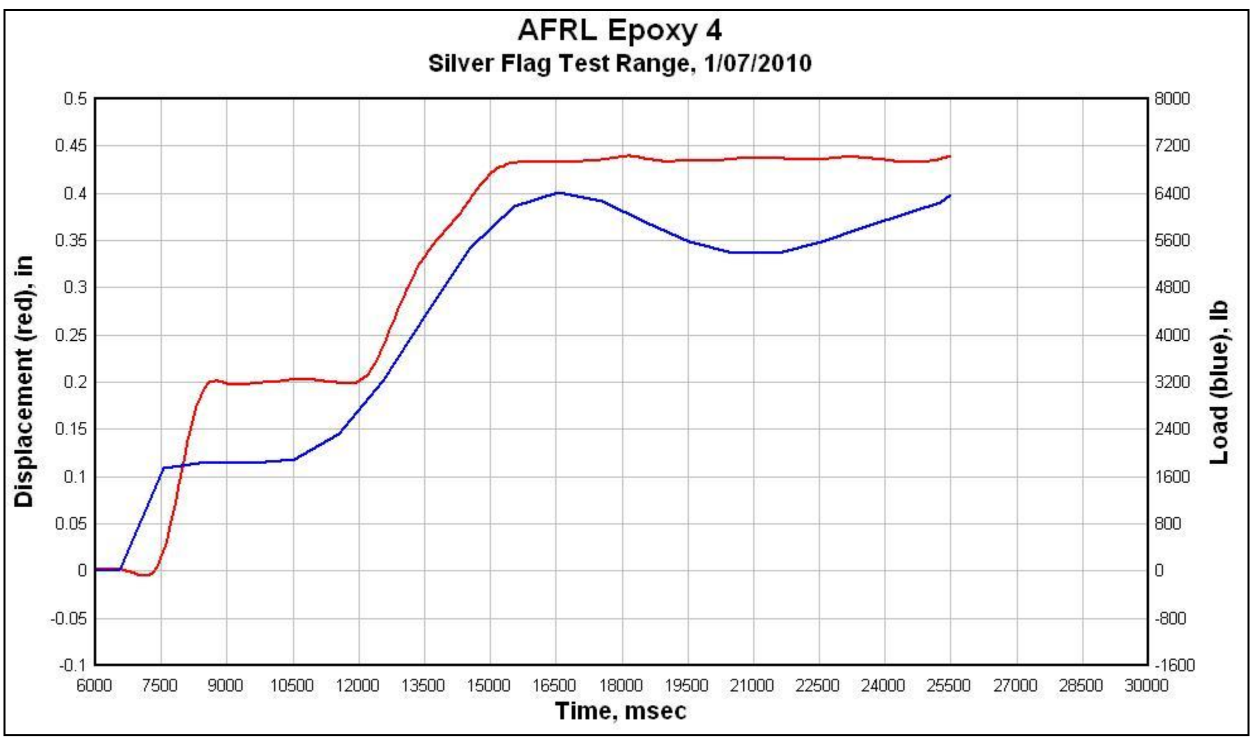
Appendix E: Silver Flag Load Cell and Deflection Data



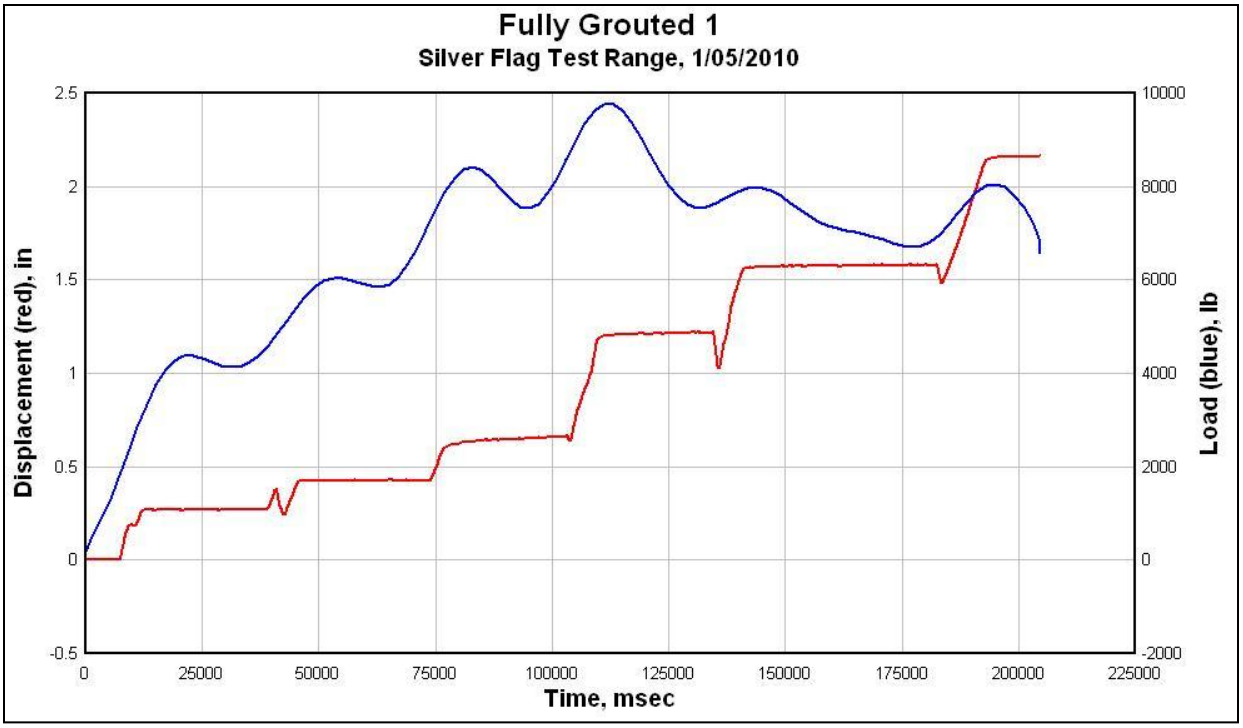


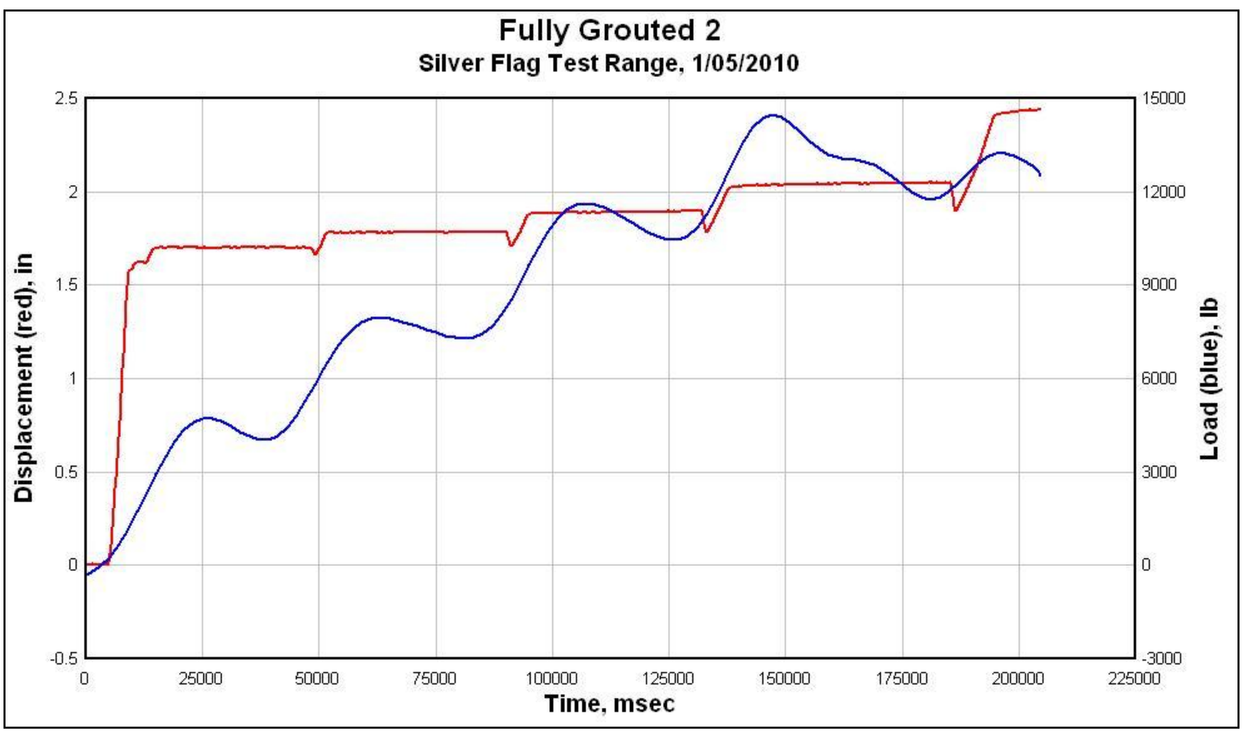


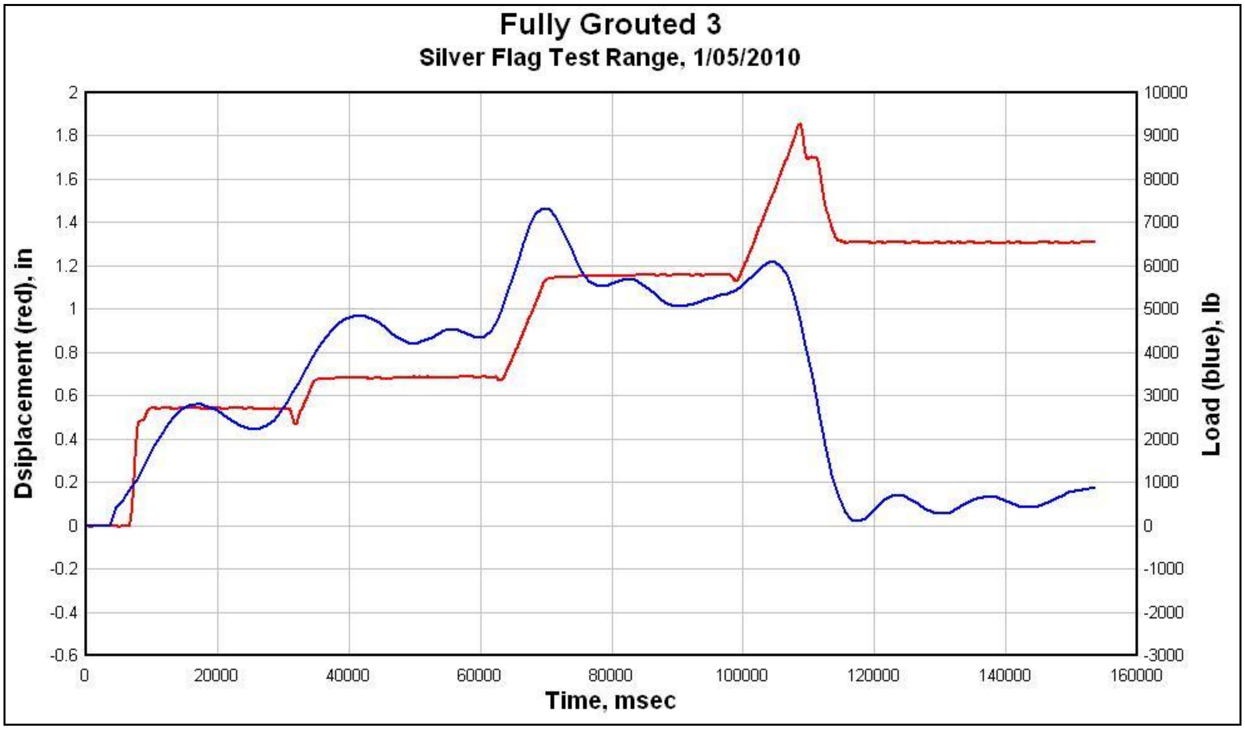
AFRL Epoxy 3
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-1000
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
50000
55000
60000
65000
70000
75000
80000

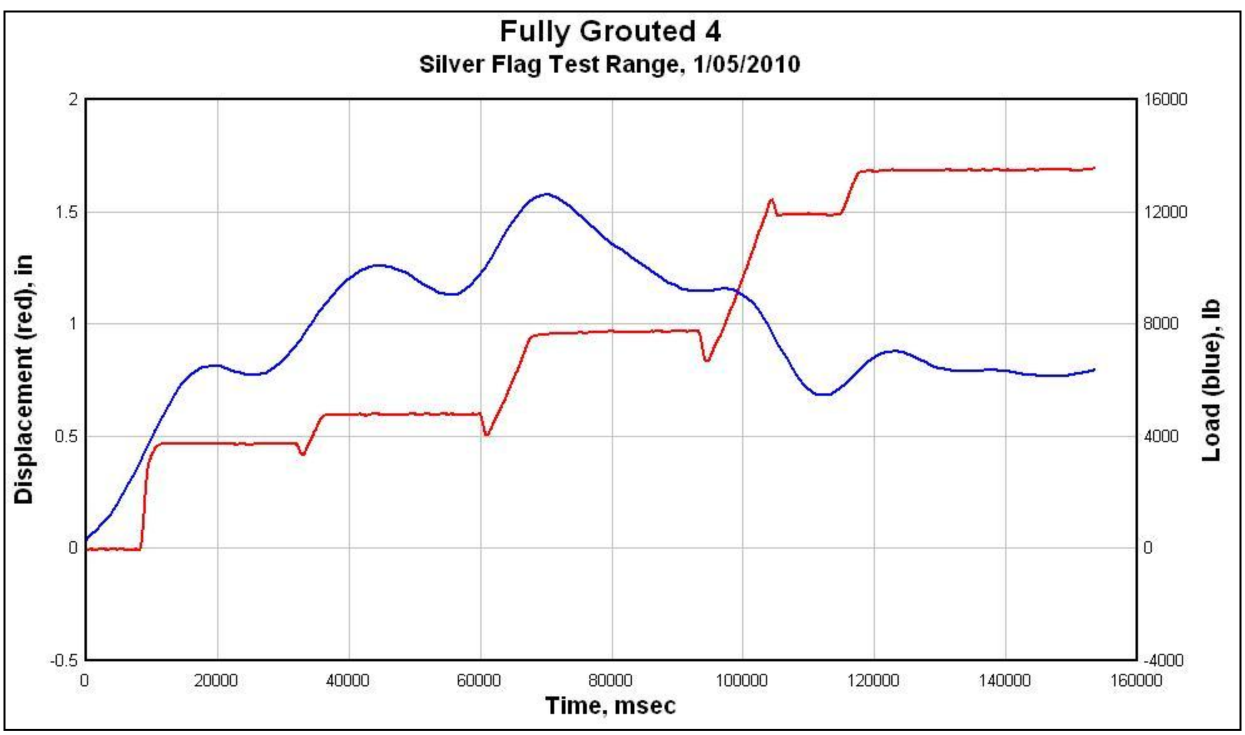


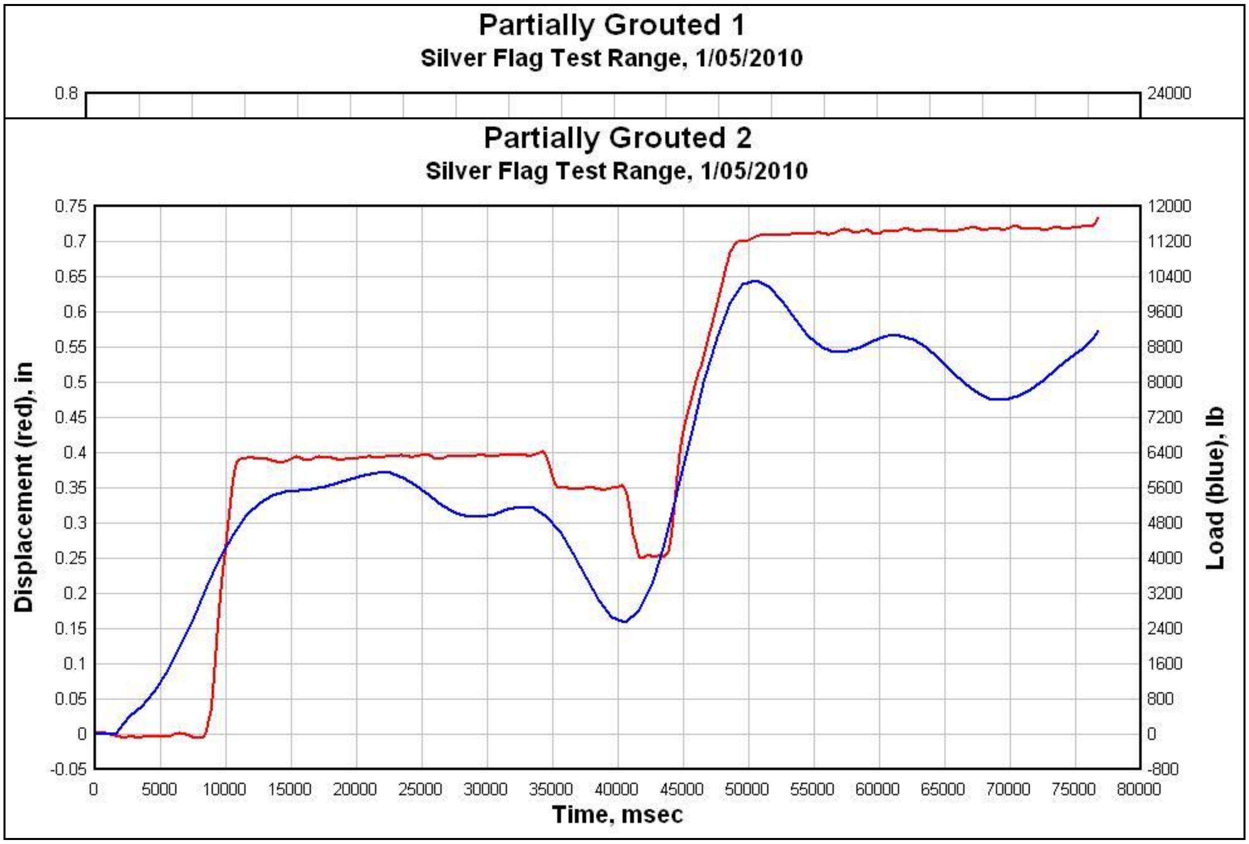
AFRL Epoxy 4
Silver Flag Test Range, 1/07/2010
Displacement (red), in
Load (blue), lb
Time, msec
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
-0.1
8000
7200
6400
5600
4800
4000
3200
2400
1600
800
0
-800
-1600
6000
7500
9000
10500
12000
13500
15000
16500
18000
19500
21000
22500
24000
25500
27000
28500
30000


Fully Grouted 1
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
2.5
2
1.5
1
0.5
0
-0.5
10000
8000
6000
4000
2000
0
-2000
0
25000
50000
75000
100000
125000
150000
175000
200000
225000

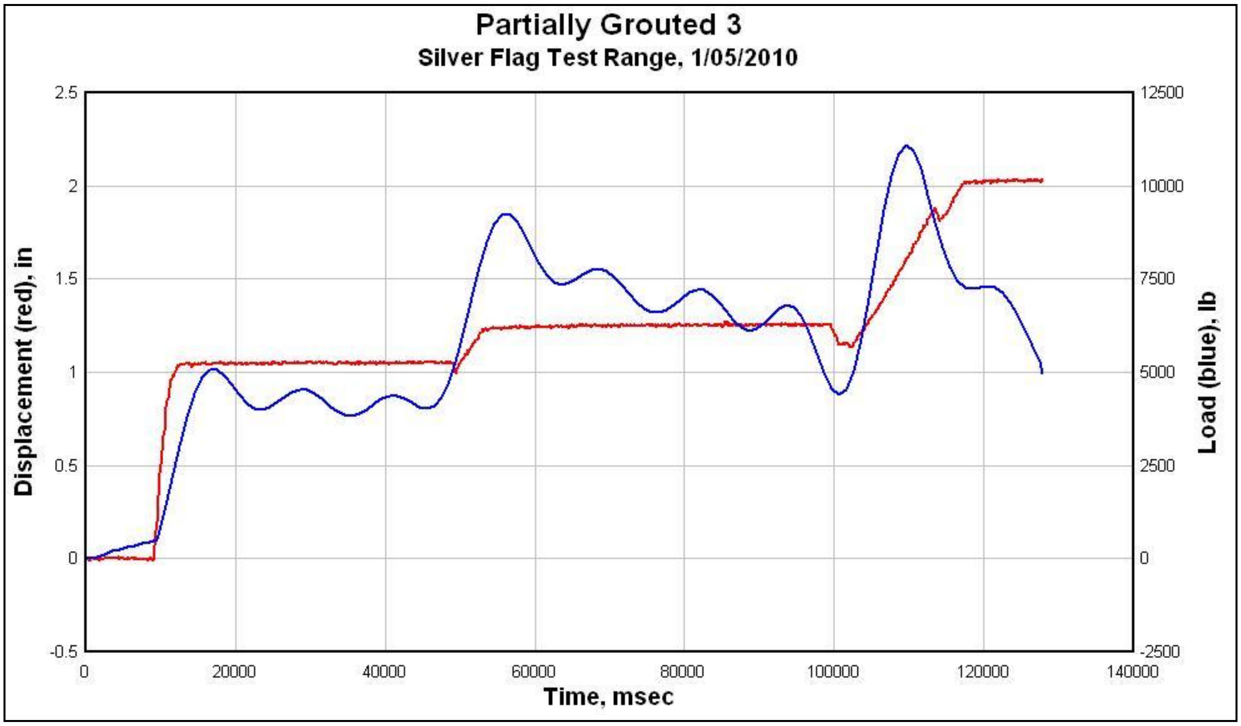

Fully Grouted 2
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
2.5
2
1.5
1
0.5
0
-0.5
15000
12000
9000
6000
3000
0
-3000
0
25000
50000
75000
100000
125000
150000
175000
200000
225000


Fully Grouted 3
Silver Flag Test Range, 1/05/2010
Dsiplacement (red), in
Load (blue), lb
Time, msec
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
0
20000
40000
60000
80000
100000
120000
140000
160000

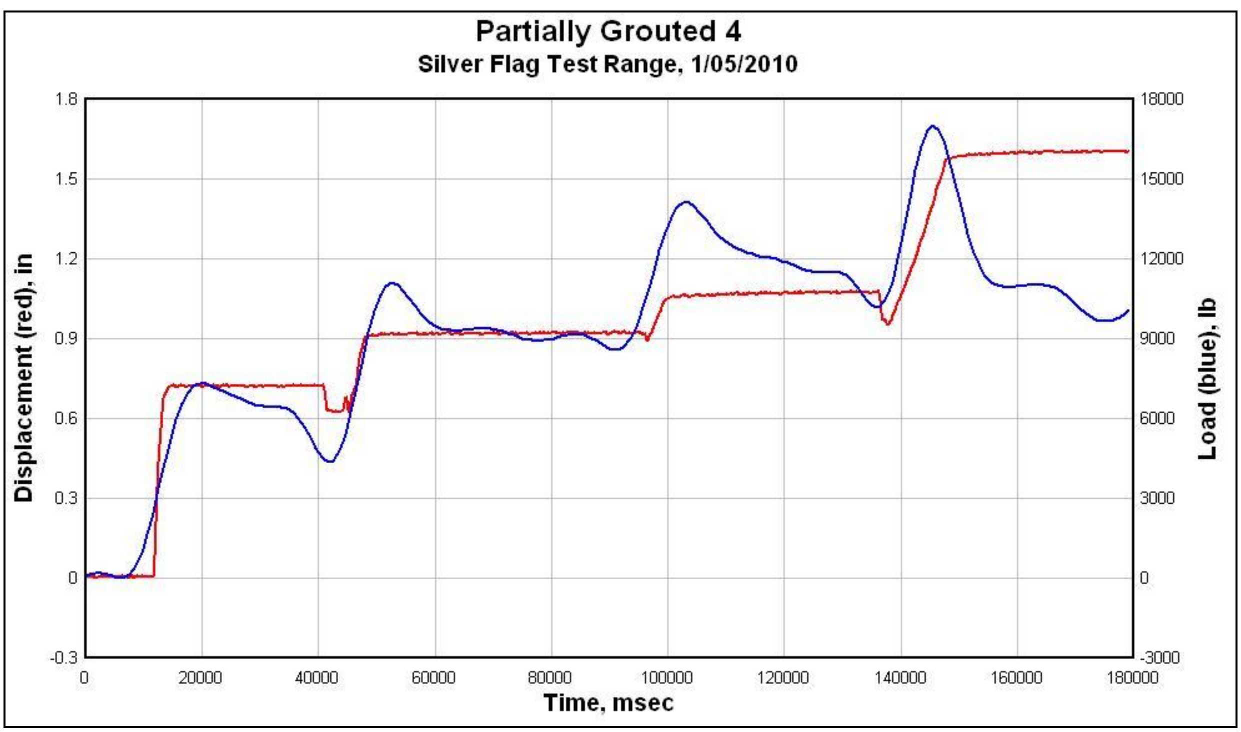

Fully Grouted 4
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
2
1.5
1
0.5
0
-0.5
16000
12000
8000
4000
0
-4000
0
20000
40000
60000
80000
100000
120000
140000
160000



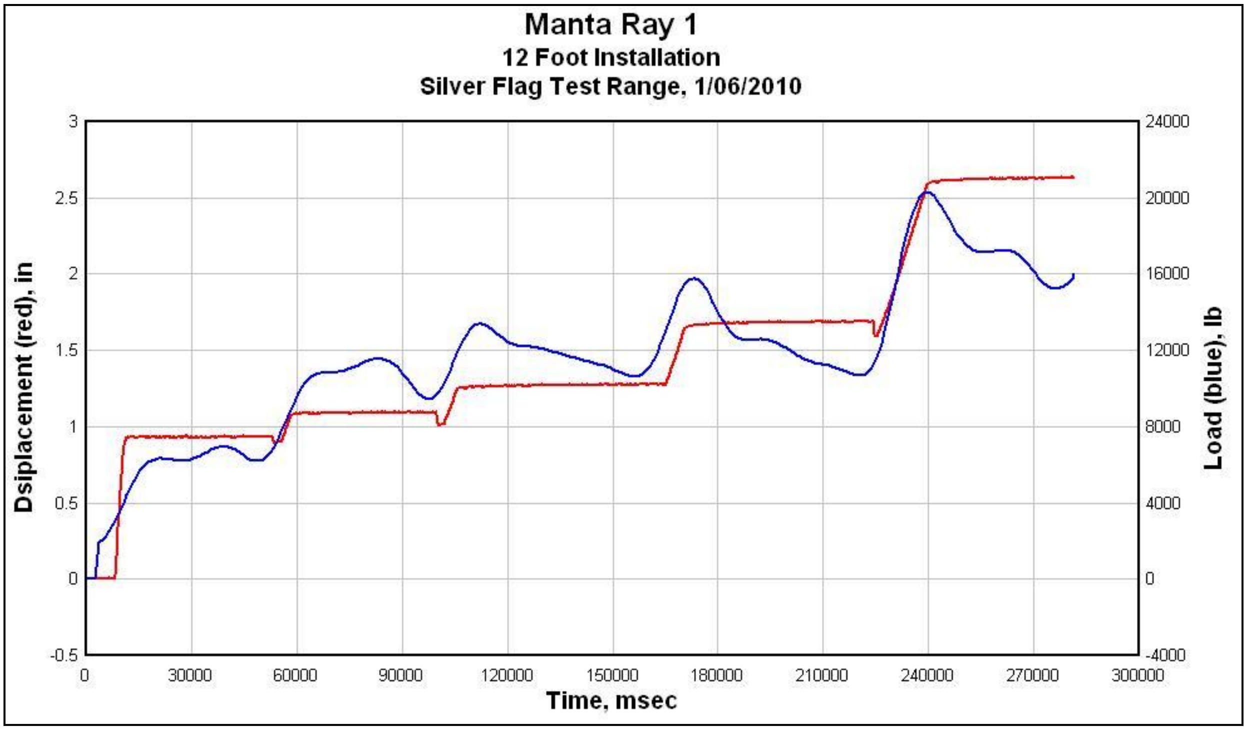
Partially Grouted 1
Silver Flag Test Range, 1/05/2010
0.8
24000
Partially Grouted 2
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
12000
11200
10400
9600
8800
8000
7200
6400
5600
4800
4000
3200
2400
1600
800
0
-800
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
50000
55000
60000
65000
70000
75000
80000

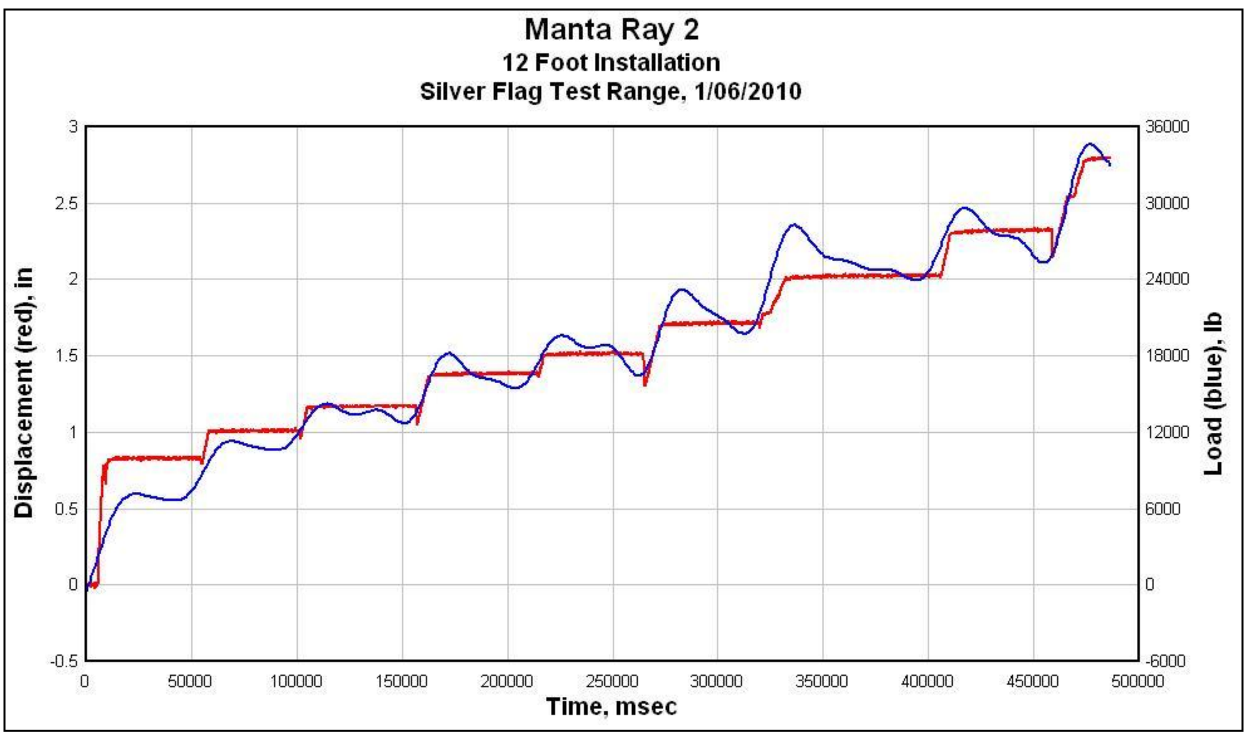


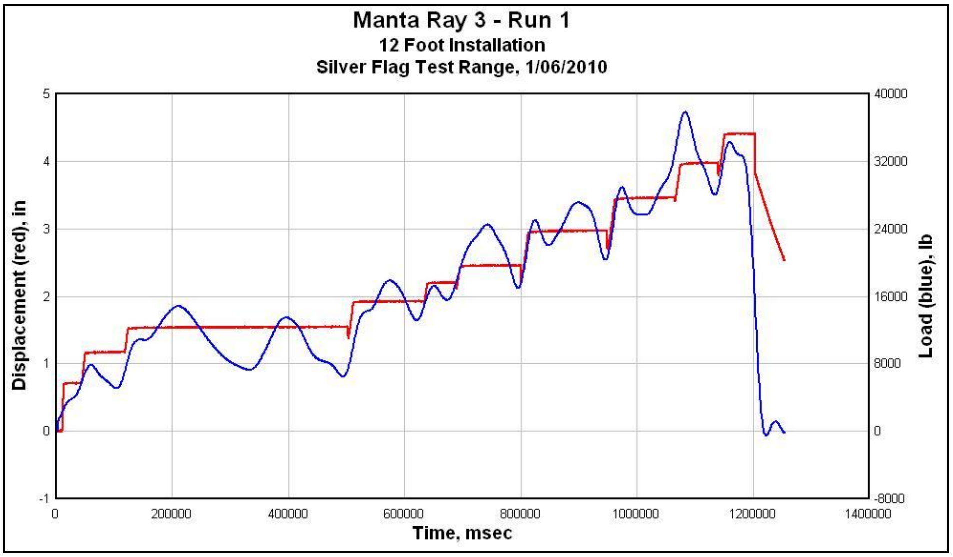
Partially Grouted 3
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
2.5
2
1.5
1
0.5
0
-0.5
12500
10000
7500
5000
2500
0
-2500
0
20000
40000
60000
80000
100000
120000
140000

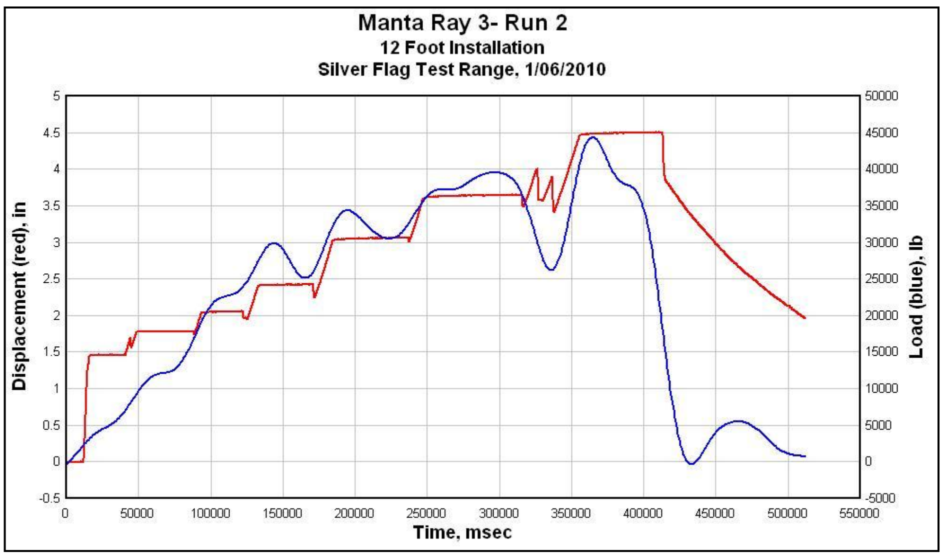


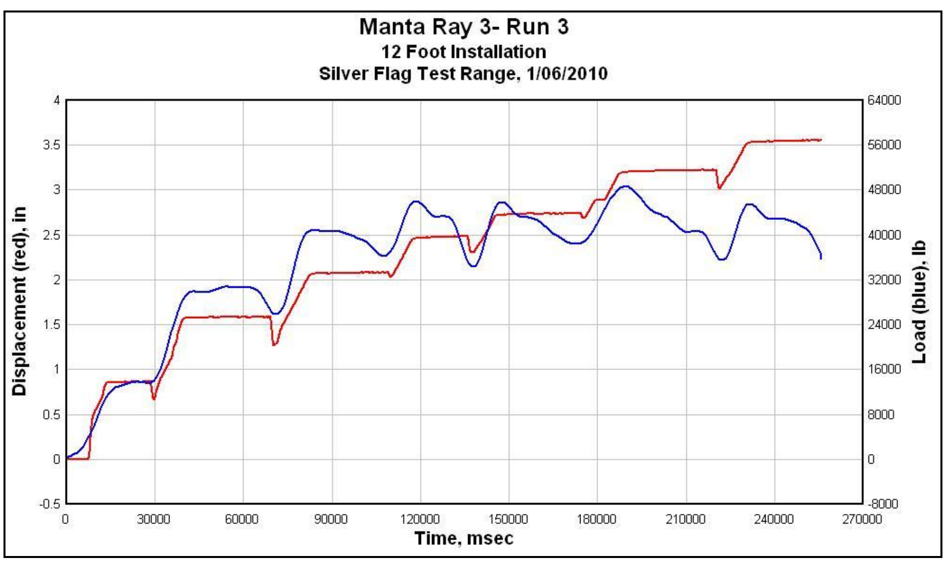
Partially Grouted 4
Silver Flag Test Range, 1/05/2010
Displacement (red), in
Load (blue), lb
Time, msec
1.8
1.5
1.2
0.9
0.6
0.3
0
-0.3
18000
15000
12000
9000
6000
3000
0
-3000
0
20000
40000
60000
80000
100000
120000
140000
160000
180000

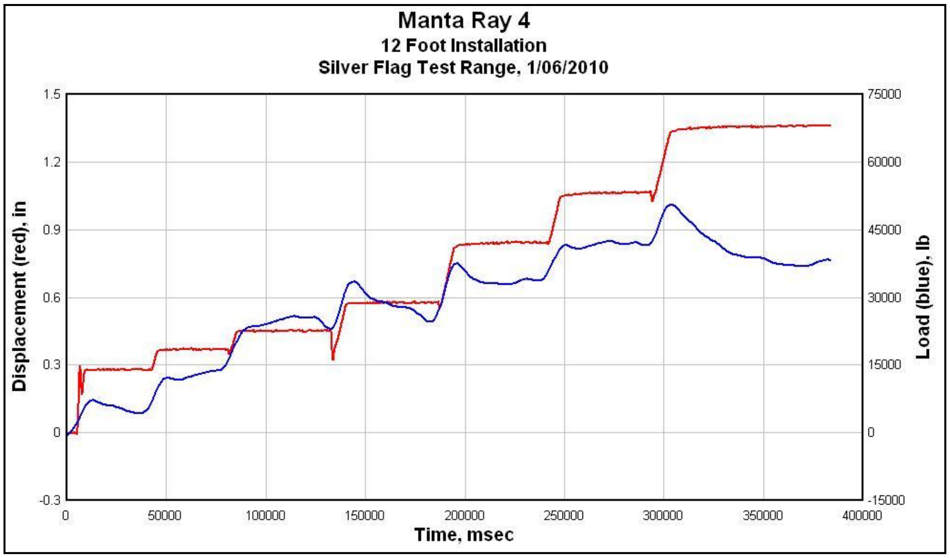

Manta Ray 1
12 Foot Installation
Silver Flag Test Range, 1/06/2010
Dsiplacement (red), in
Load (blue), lb
Time, msec
3
2.5
2
1.5
1
0.5
0
-0.5
24000
20000
16000
12000
8000
4000
0
-4000
0
30000
60000
90000
120000
150000
180000
210000
240000
270000
300000

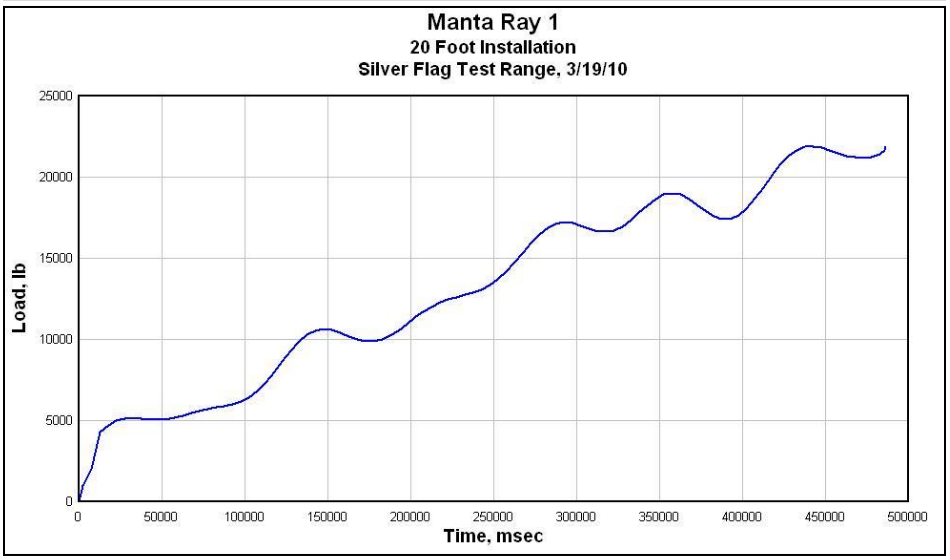

Manta Ray 2
12 Foot Installation
Silver Flag Test Range, 1/06/2010
Displacement (red), in
Load (blue), lb
Time, msec
3
2.5
2
1.5
1
0.5
0
-0.5
36000
30000
24000
18000
12000
6000
0
-6000
0
50000
100000
150000
200000
250000
300000
350000
400000
450000
500000

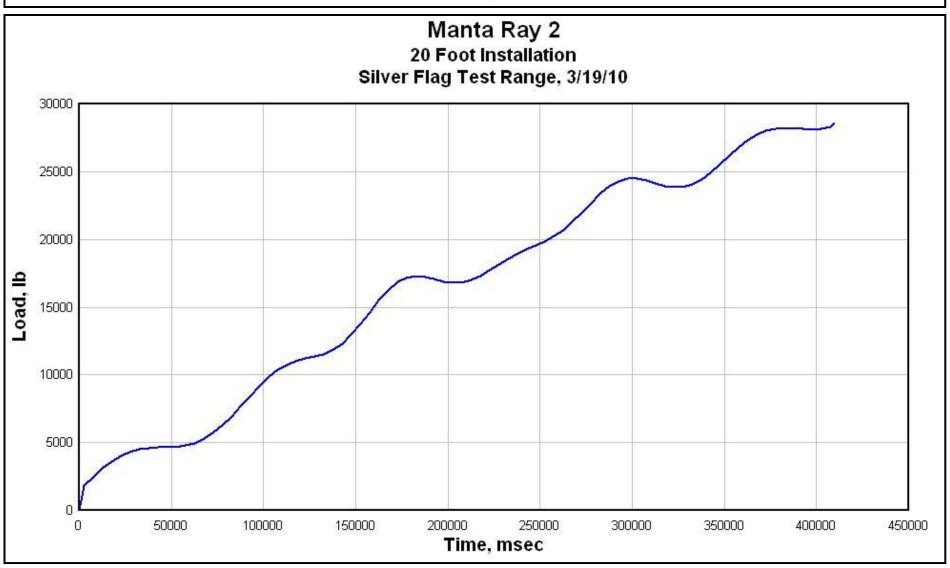

Manta Ray 3 - Run 1
12 Foot Installation
Silver Flag Test Range, 1/06/2010
Displacement (red), in
Load (blue), lb
Time, msec

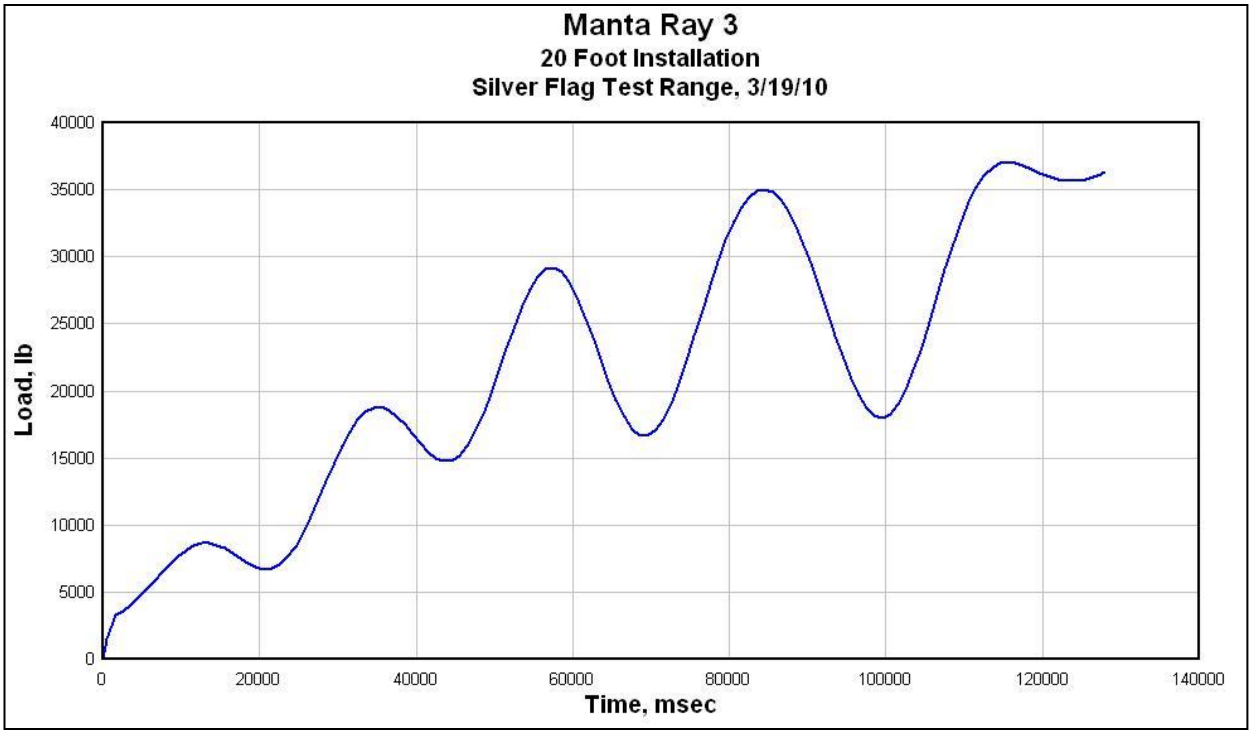

Manta Ray 3- Run 2
12 Foot Installation
Silver Flag Test Range, 1/06/2010
Displacement (red), in
Load (blue), lb
Time, msec


Manta Ray 3- Run 3
12 Foot Installation
Silver Flag Test Range, 1/06/2010
Displacement (red), in
Load (blue), lb
Time, msec

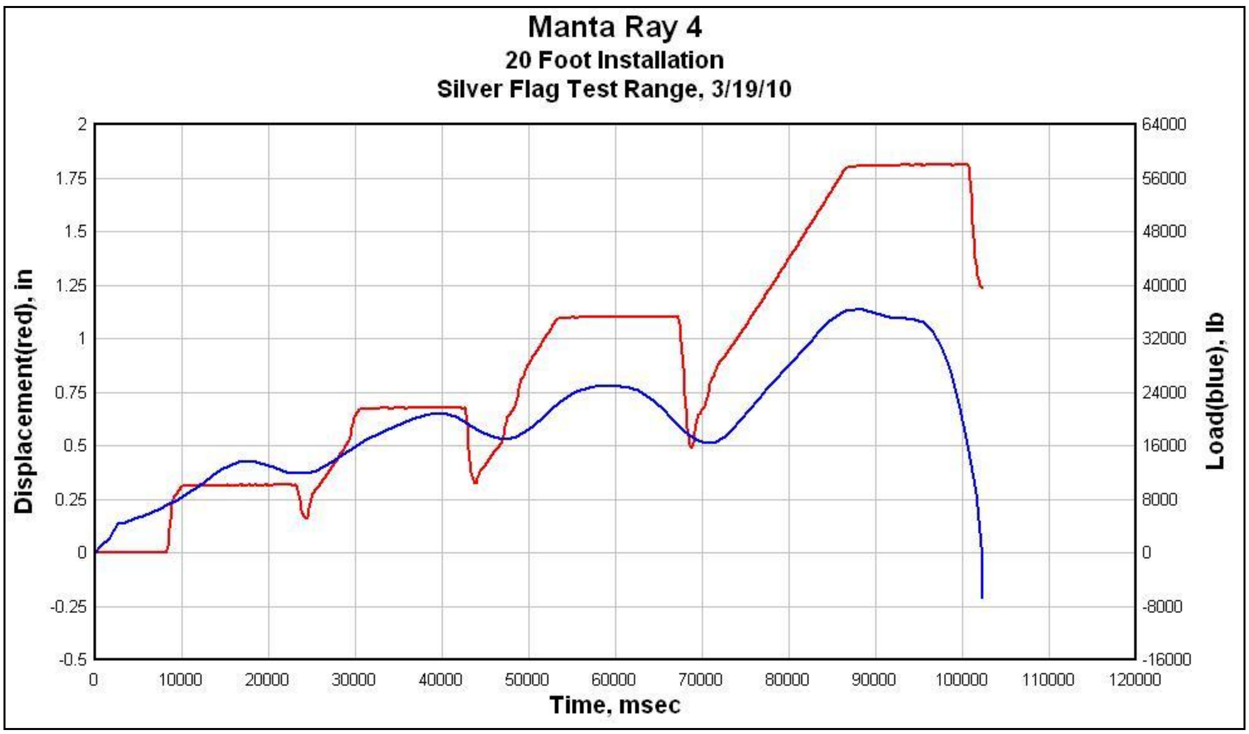

Manta Ray 4
12 Foot Installation
Silver Flag Test Range, 1/06/2010
Displacement (red), in
Load (blue), lb
Time, msec


Manta Ray 1
20 Foot Installation
Silver Flag Test Range, 3/19/10
Load, lb
Time, msec


Manta Ray 2
20 Foot Installation
Silver Flag Test Range, 3/19/10
Load, lb
Time, msec

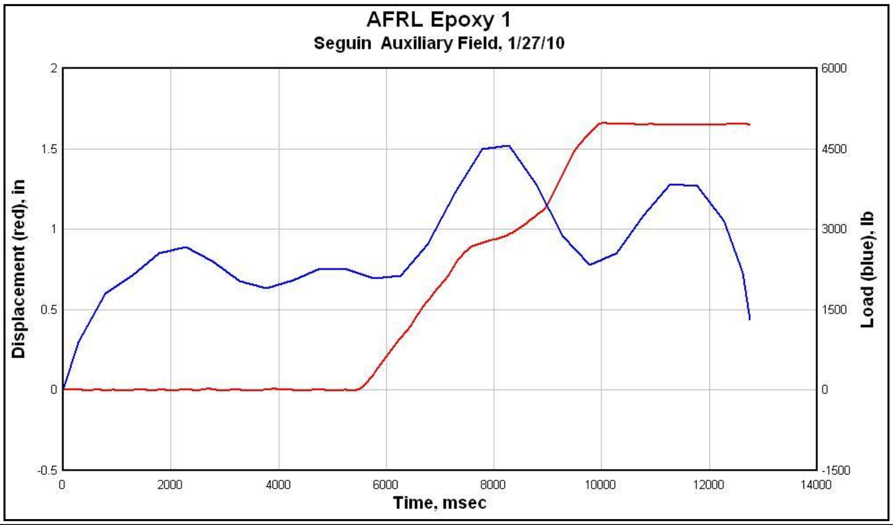


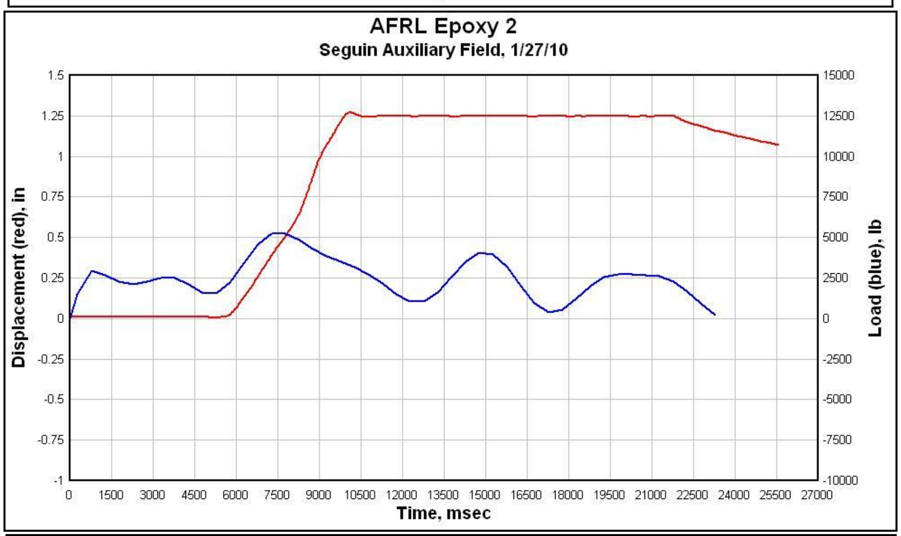
Manta Ray 3
20 Foot Installation
Silver Flag Test Range, 3/19/10
Load, lb
Time, msec
0
5000
10000
15000
20000
25000
30000
35000
40000
0
20000
40000
60000
80000
100000
120000
140000

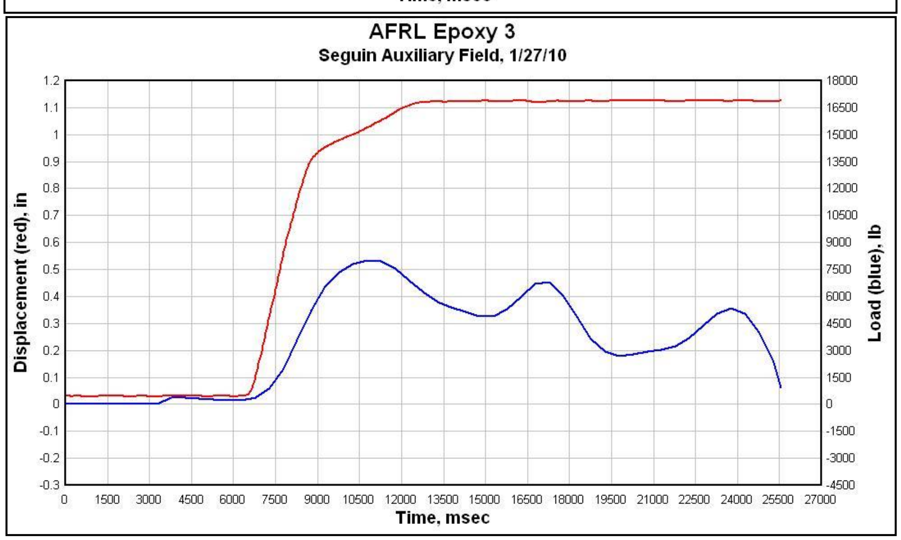


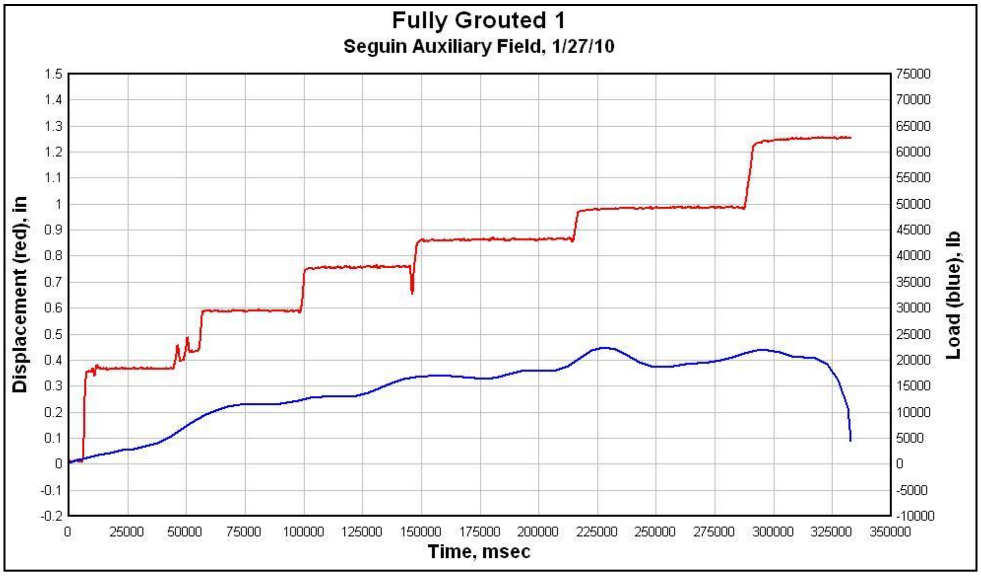
Manta Ray 4
20 Foot Installation
Silver Flag Test Range, 3/19/10
Displacement(red), in
Load(blue), lb
Time, msec
-0.5
-0.25
0
0.25
0.5
0.75
1
1.25
1.5
1.75
2
-16000
-8000
0
8000
16000
24000
32000
40000
48000
56000
64000
0
10000
20000
30000
40000
50000
60000
70000
80000
90000
100000
110000
120000

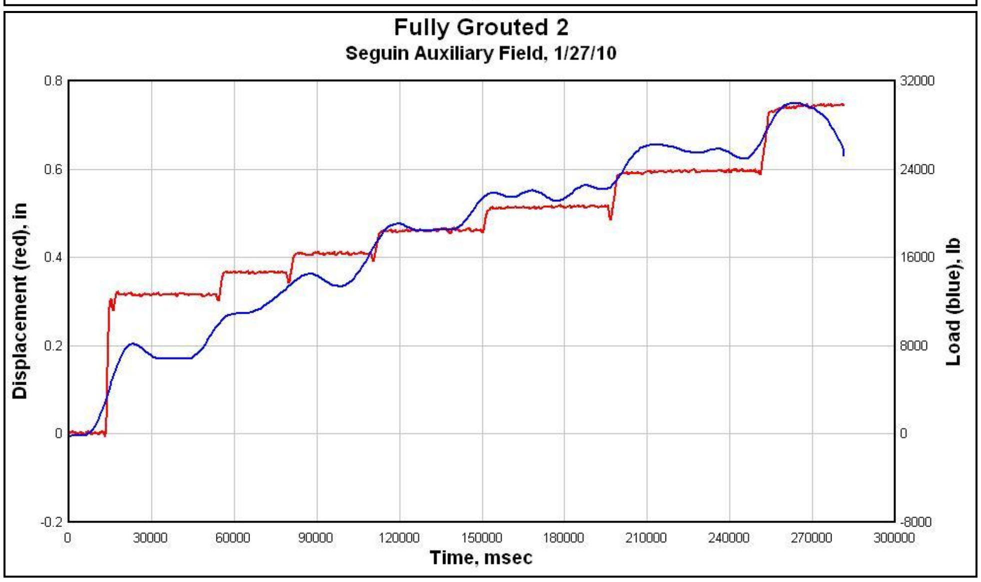
Appendix F: Seguin Load Cell and Deflection Data

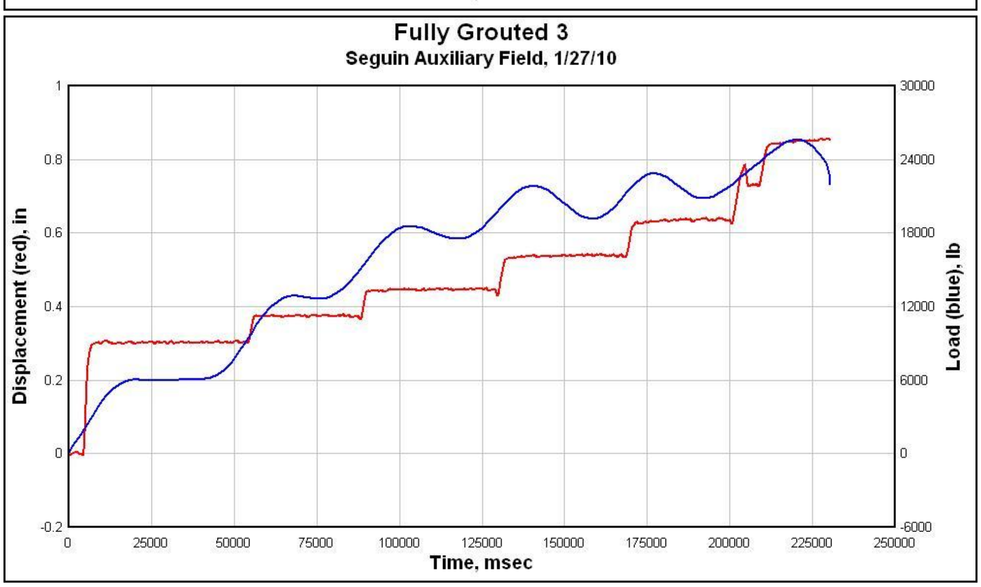


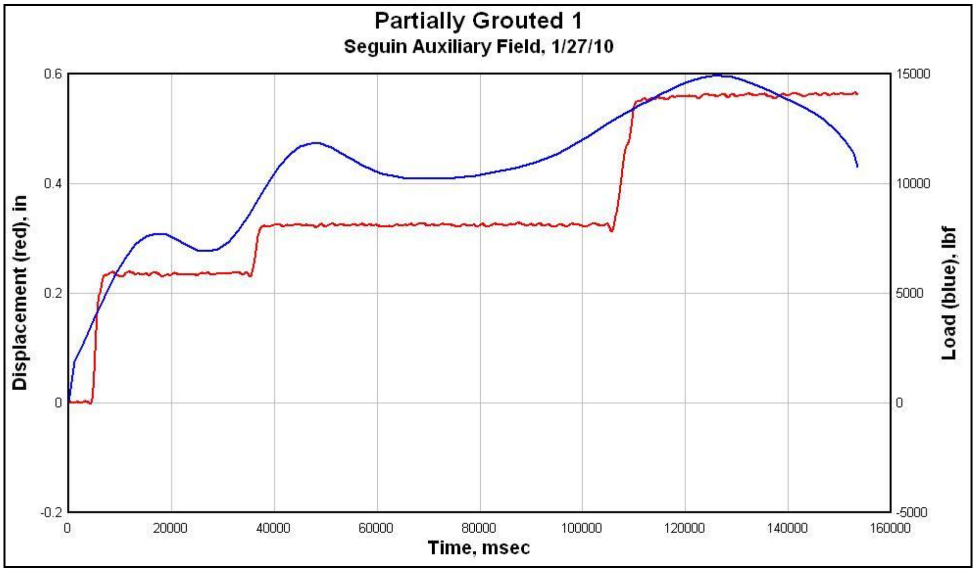




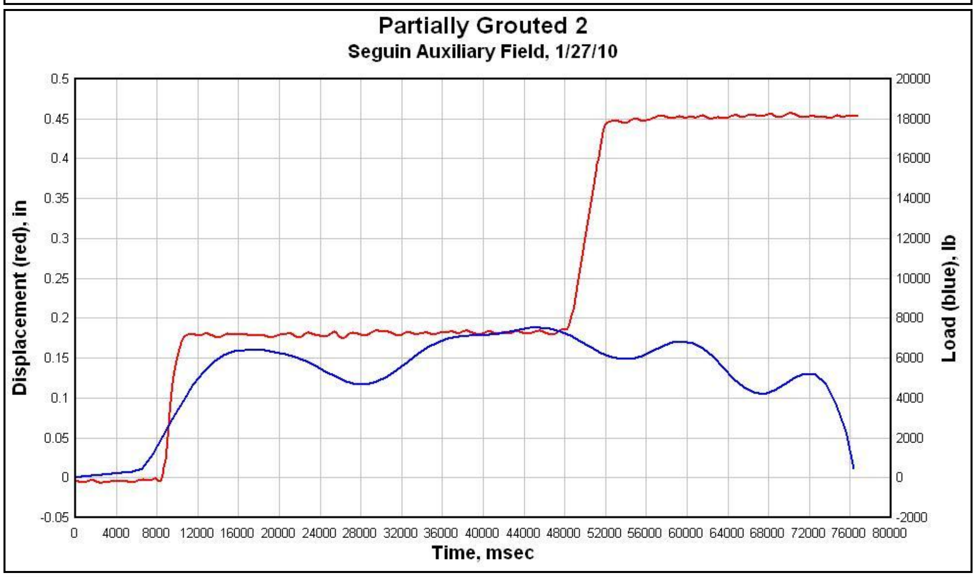

Fully Grouted 1
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec


Fully Grouted 2
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec

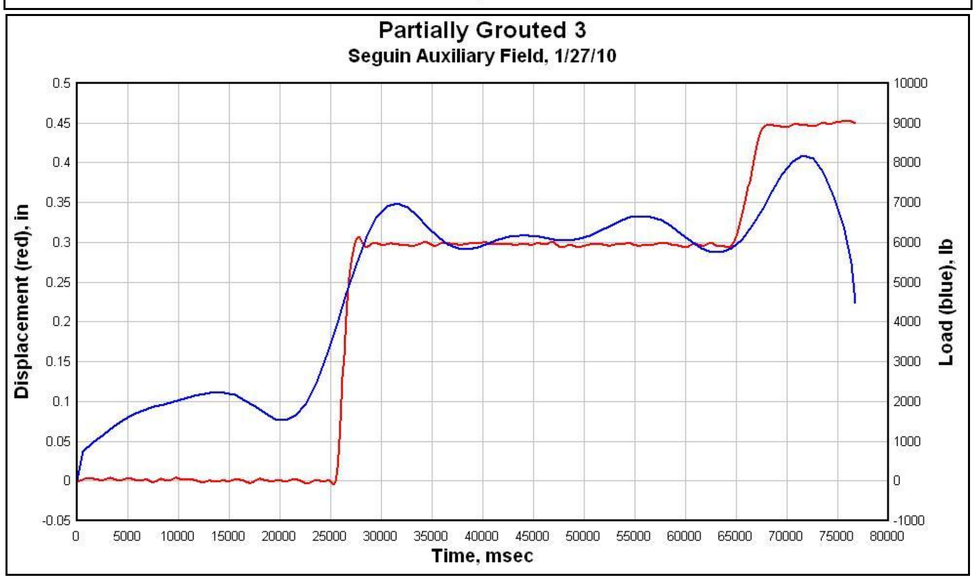

Fully Grouted 3
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec



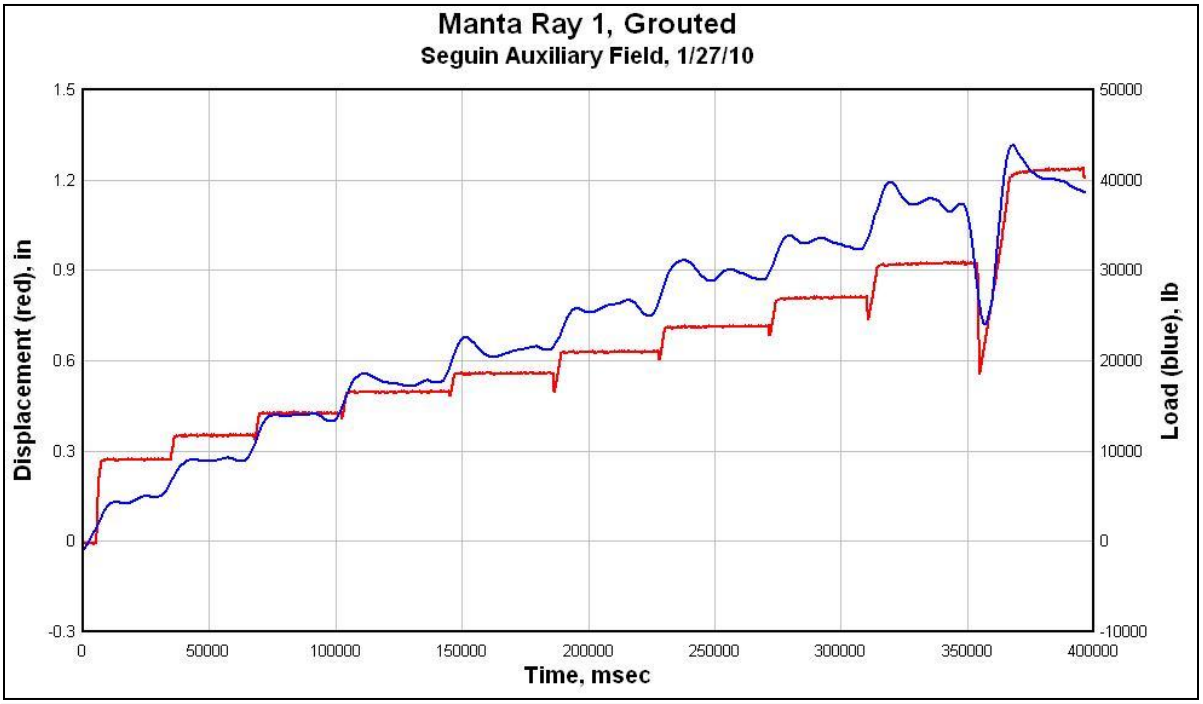
Partially Grouted 1
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lbf
Time, msec

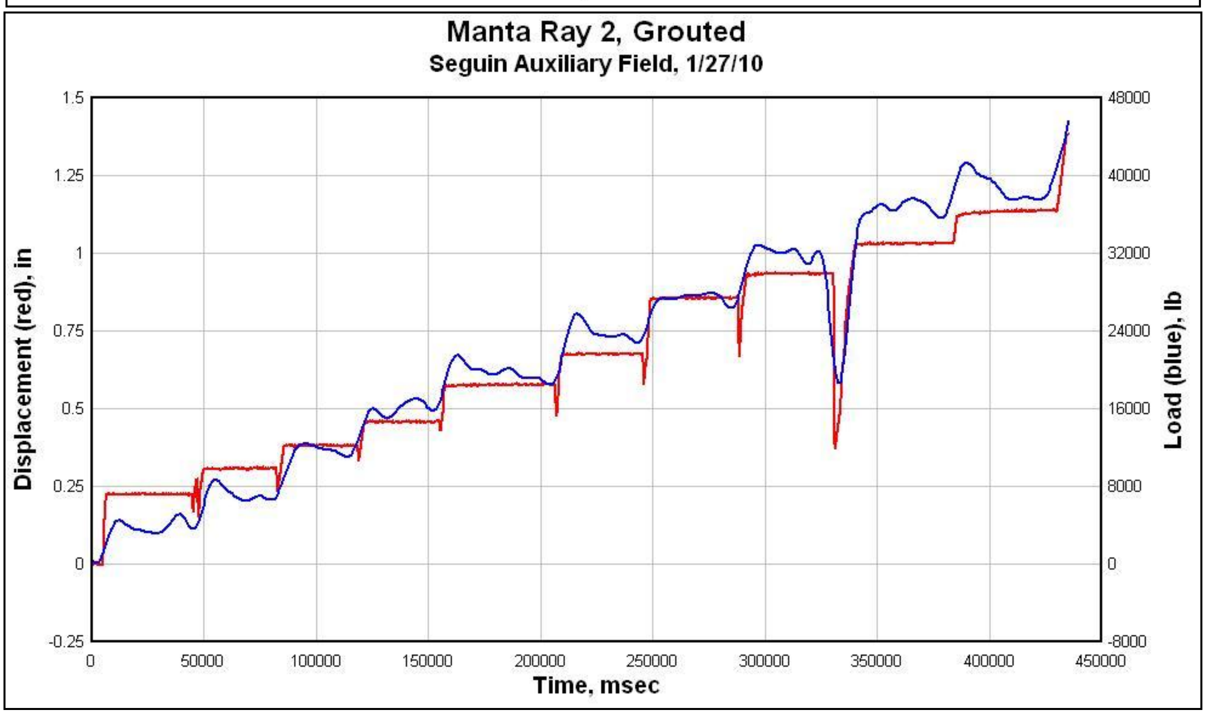


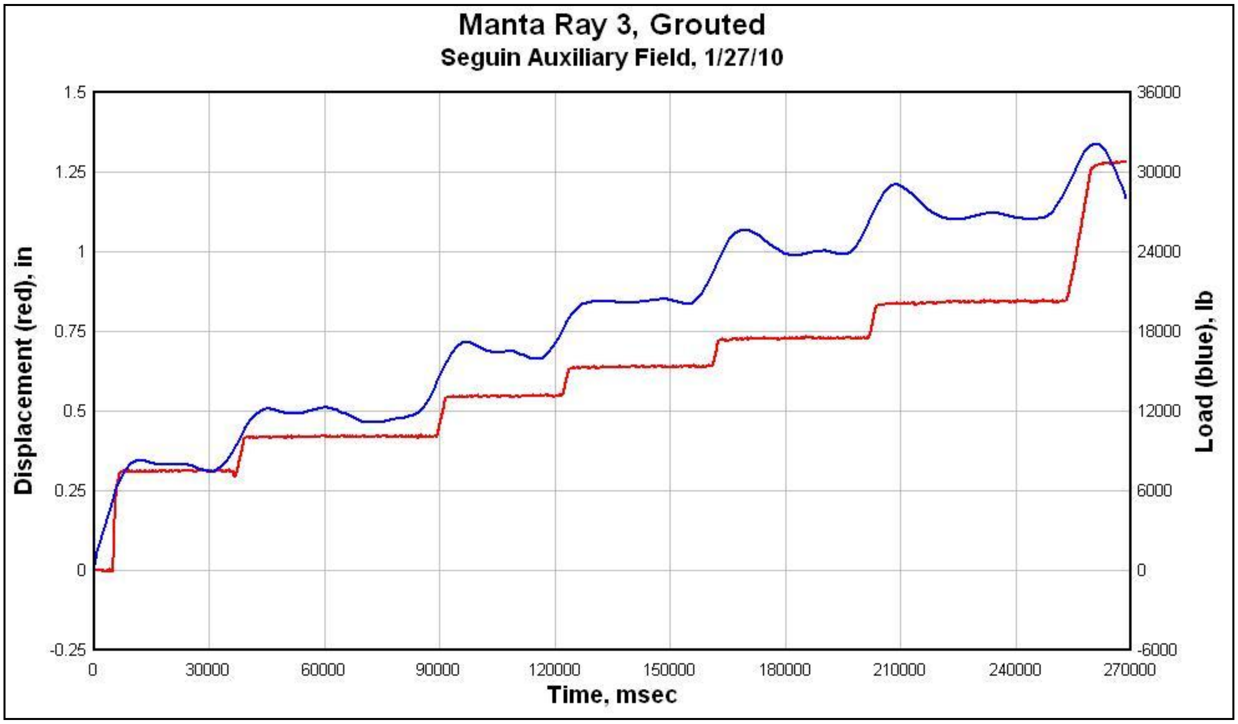
Partially Grouted 2
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec

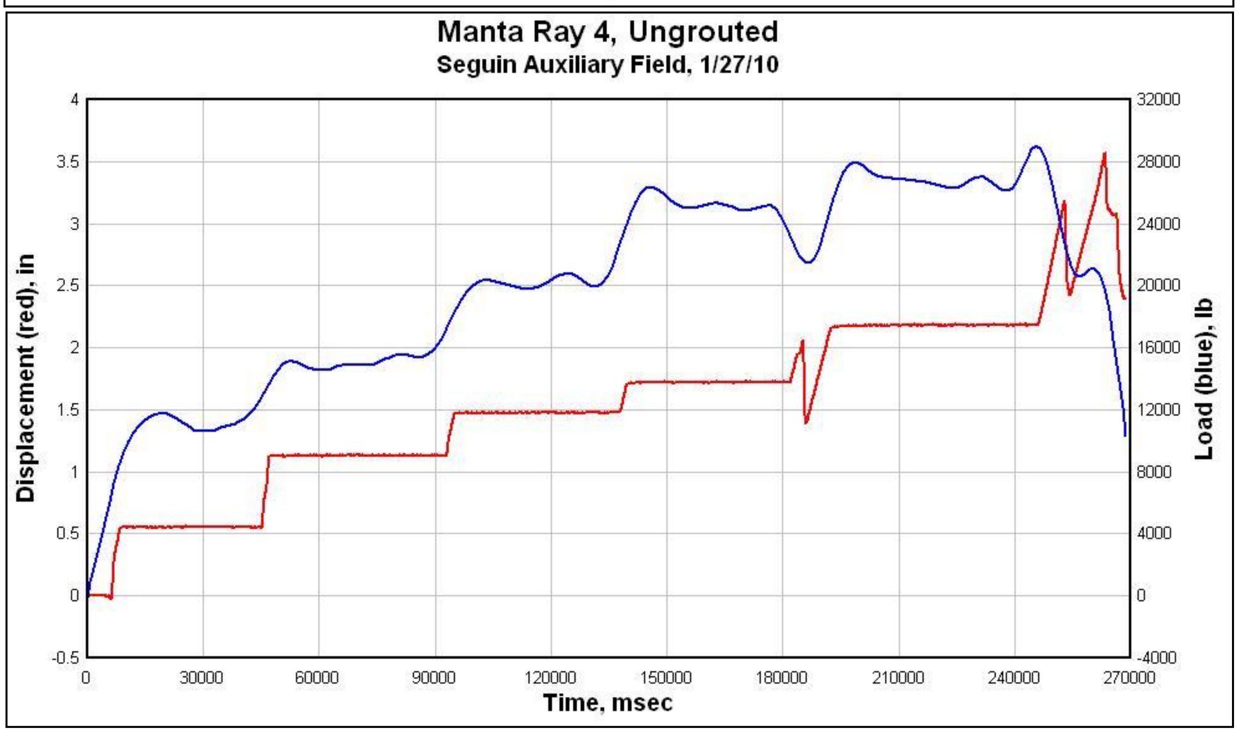


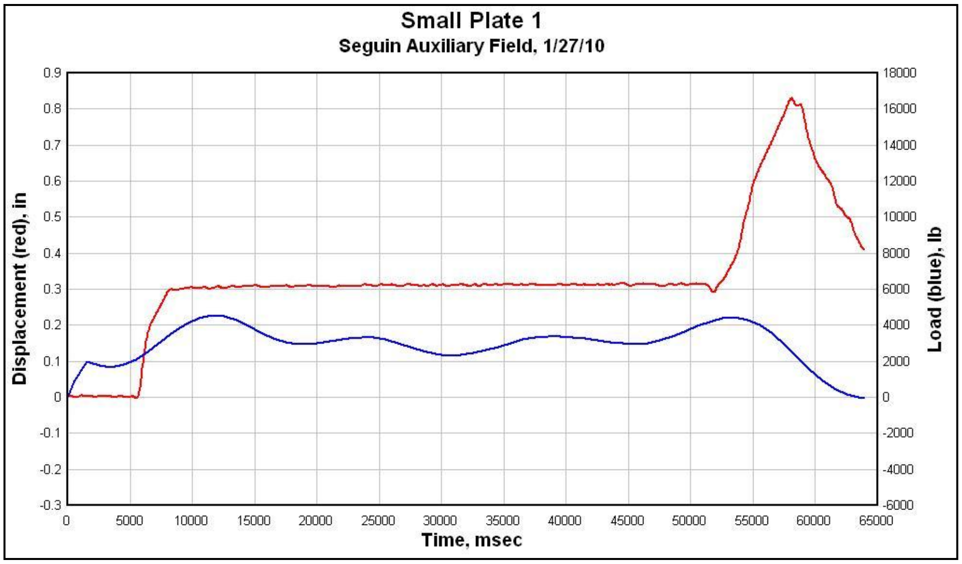
Partially Grouted 3
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec

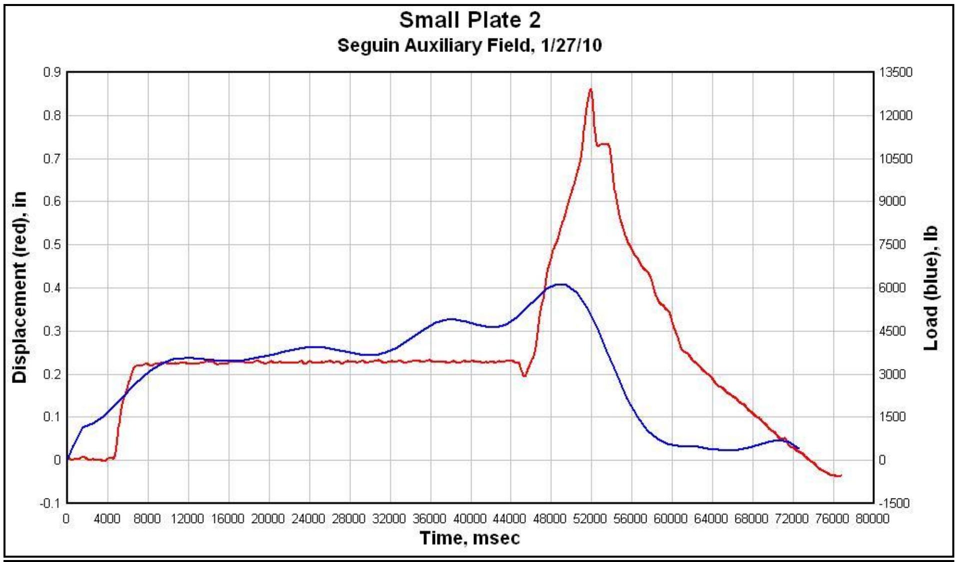

Manta Ray 1, Grouted
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
1.5
1.2
0.9
0.6
0.3
0
-0.3
50000
40000
30000
20000
10000
0
-10000
0
50000
100000
150000
200000
250000
300000
350000
400000


Manta Ray 2, Grouted
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
1.5
1.25
1
0.75
0.5
0.25
0
-0.25
48000
40000
32000
24000
16000
8000
0
-8000
0
50000
100000
150000
200000
250000
300000
350000
400000
450000

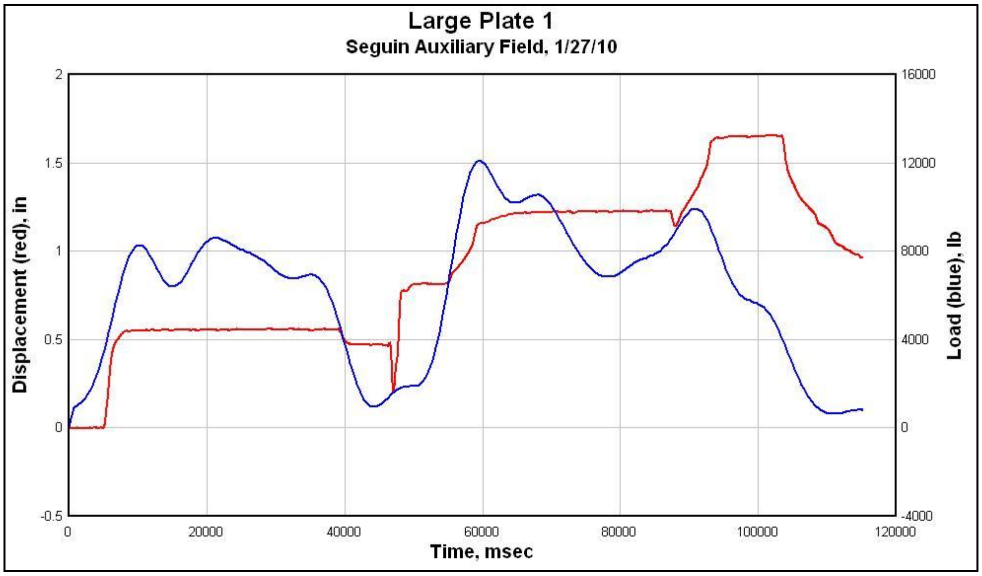

Manta Ray 3, Grouted
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
1.5
1.25
1
0.75
0.5
0.25
0
-0.25
36000
30000
24000
18000
12000
6000
0
-6000
0
30000
60000
90000
120000
150000
180000
210000
240000
270000

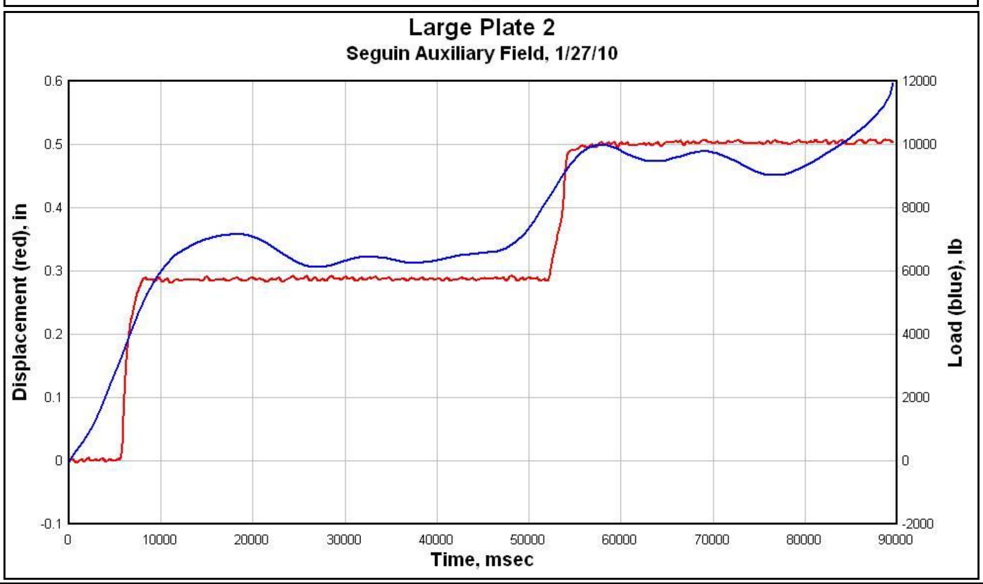

Manta Ray 4, Ungrouted
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
4
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
32000
28000
24000
20000
16000
12000
8000
4000
0
-4000
0
30000
60000
90000
120000
150000
180000
210000
240000
270000

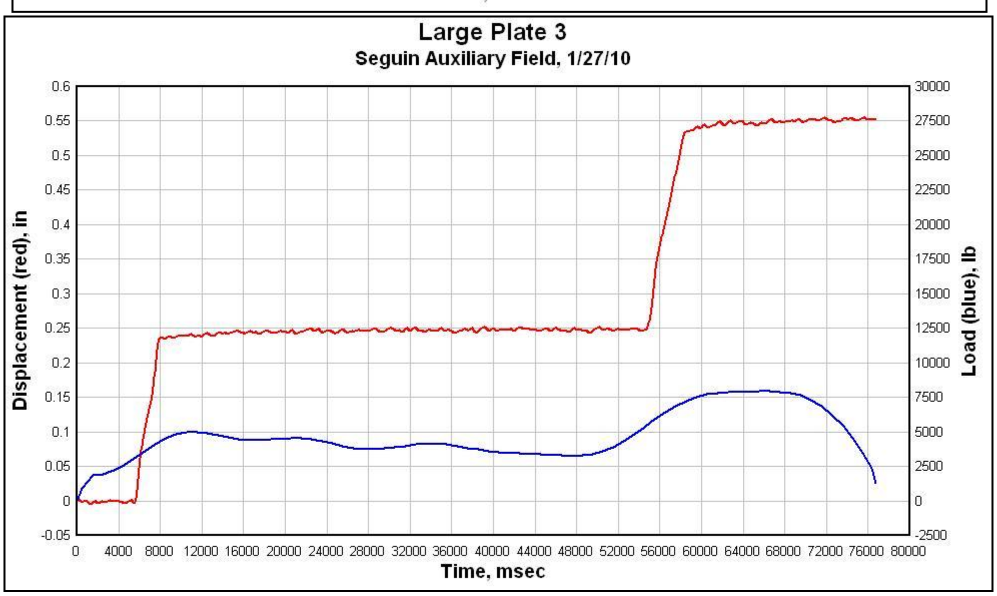

Small Plate 1
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec


Small Plate 2
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec

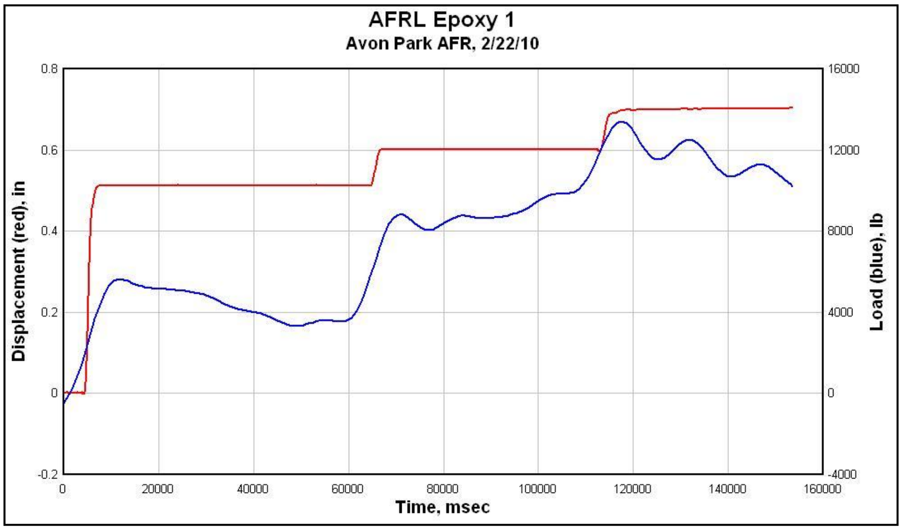

Small Plate 3
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec

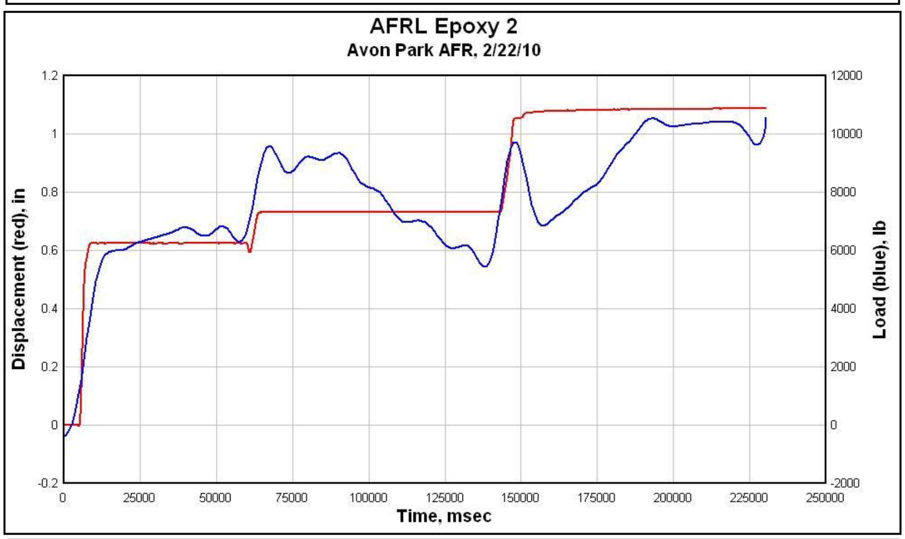

Large Plate 1
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
2
1.5
1
0.5
0
-0.5
16000
12000
8000
4000
0
-4000
0
20000
40000
60000
80000
100000
120000

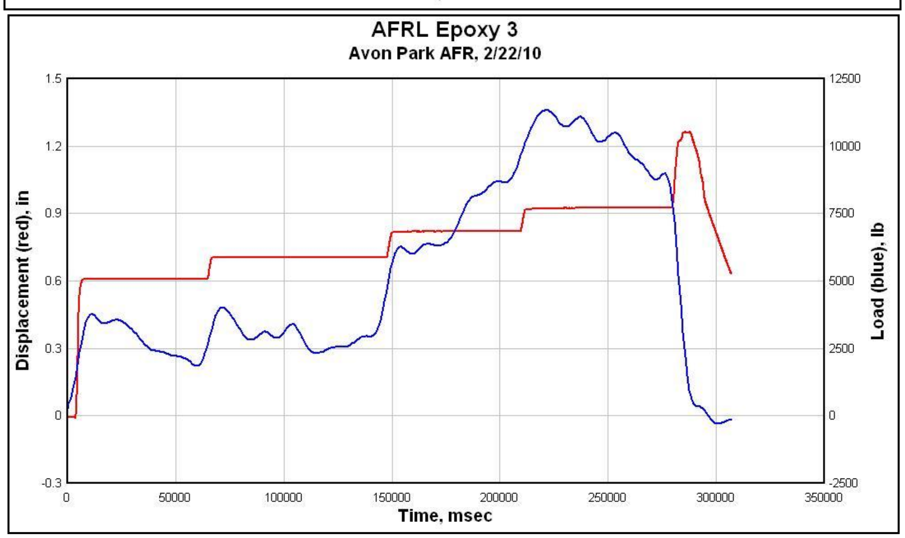

Large Plate 2
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
12000
10000
8000
6000
4000
2000
0
-2000
0
10000
20000
30000
40000
50000
60000
70000
80000
90000

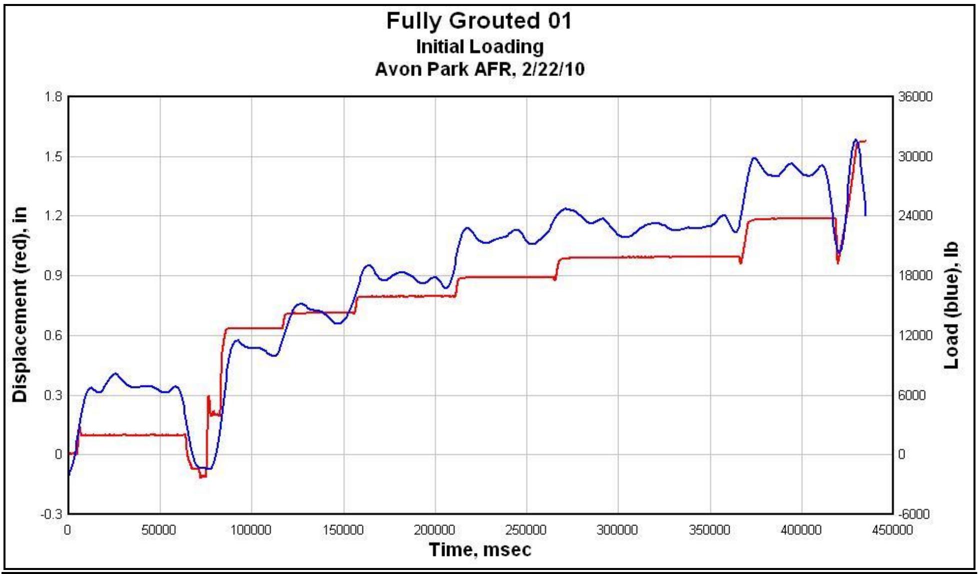

Large Plate 3
Seguin Auxiliary Field, 1/27/10
Displacement (red), in
Load (blue), lb
Time, msec
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
30000
27500
25000
22500
20000
17500
15000
12500
10000
7500
5000
2500
0
-2500
0
4000
8000
12000
16000
20000
24000
28000
32000
36000
40000
44000
48000
52000
56000
60000
64000
68000
72000
76000
80000

Appendix G
Avon Park Load Cell and Deflection Data



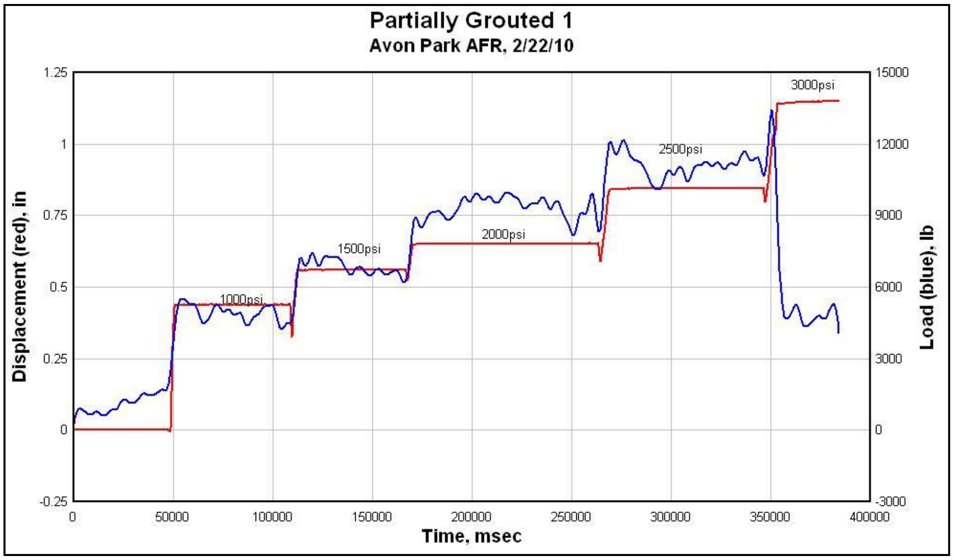


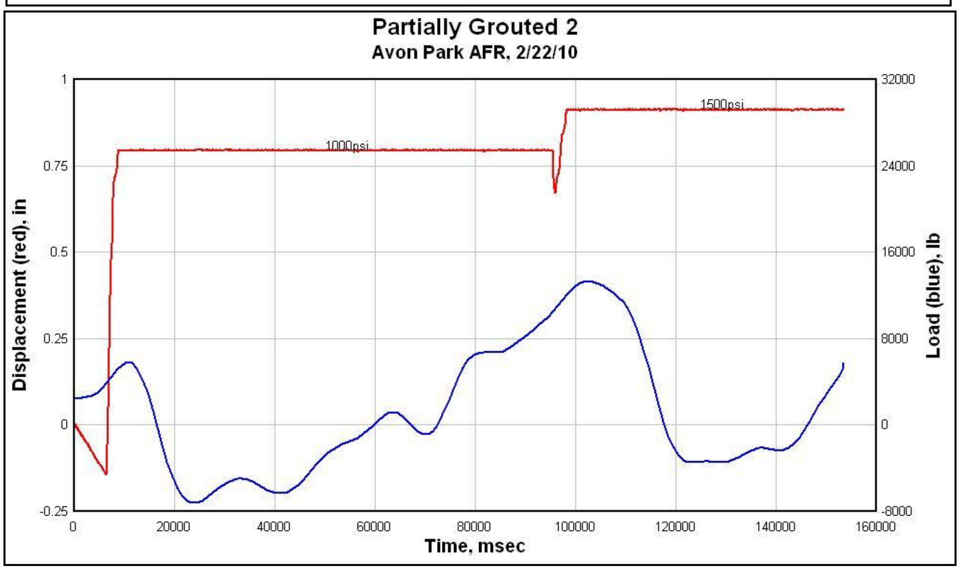


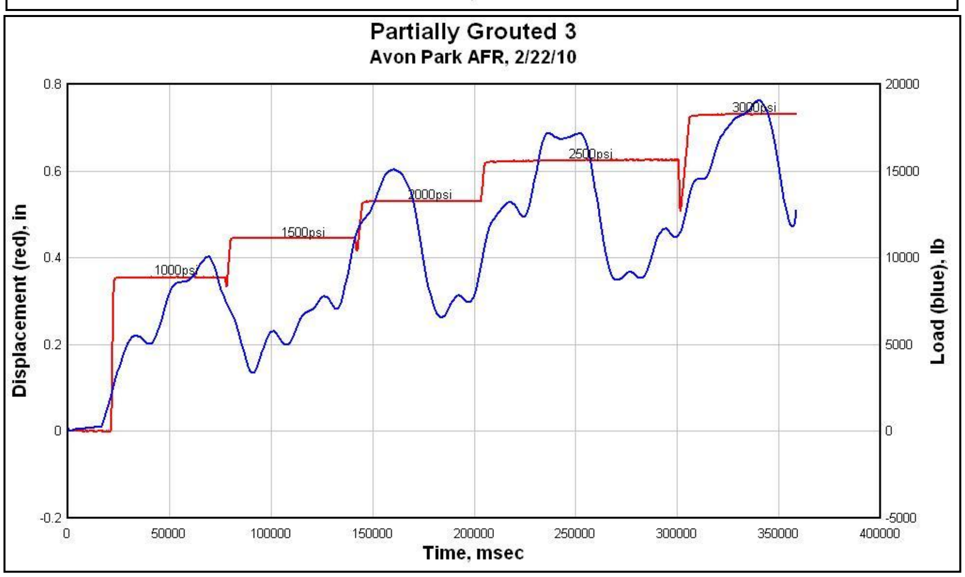

Fully Grouted 01
Initial Loading
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1.8
1.5
1.2
0.9
0.6
0.3
0
-0.3
36000
30000
24000
18000
12000
6000
0
-6000
0
50000
100000
150000
200000
250000
300000
350000
400000
450000

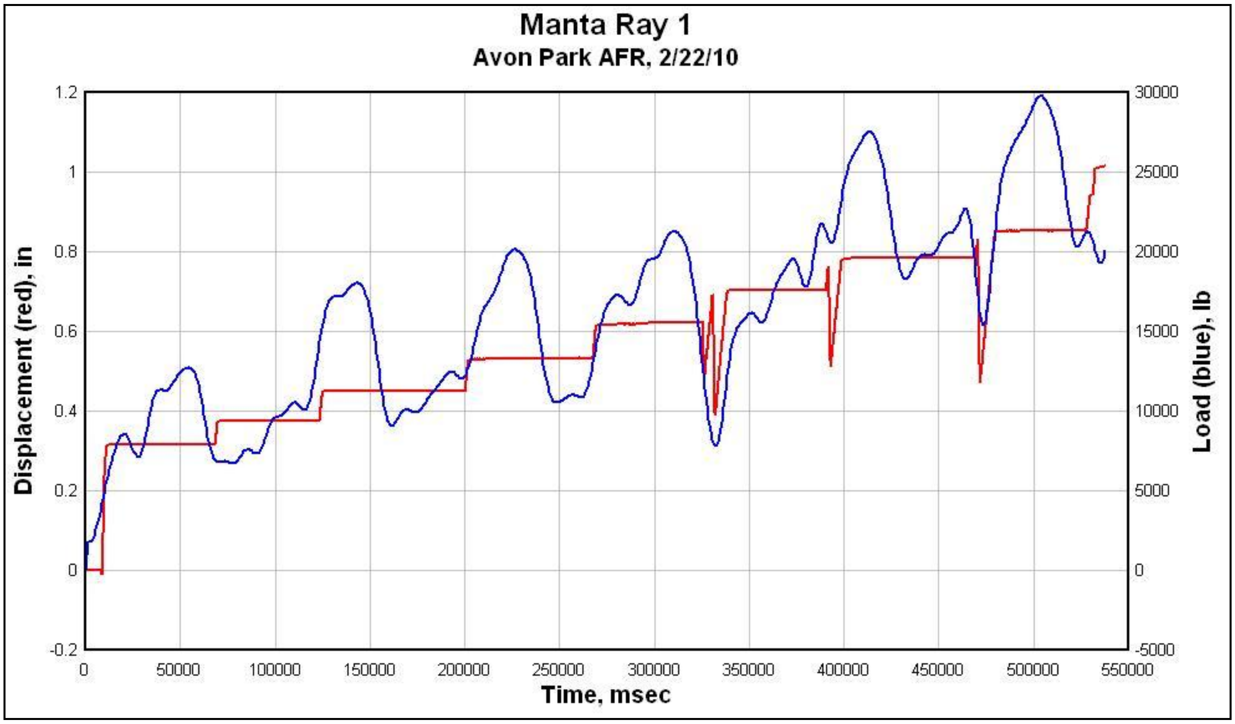

Fully Grouted 1a
Reloading
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1.8
1.5
1.2
0.9
0.6
0.3
0
-0.3
30000
25000
20000
15000
10000
5000
0
-5000
0
3000
6000
9000
12000
15000
18000
21000
24000
27000

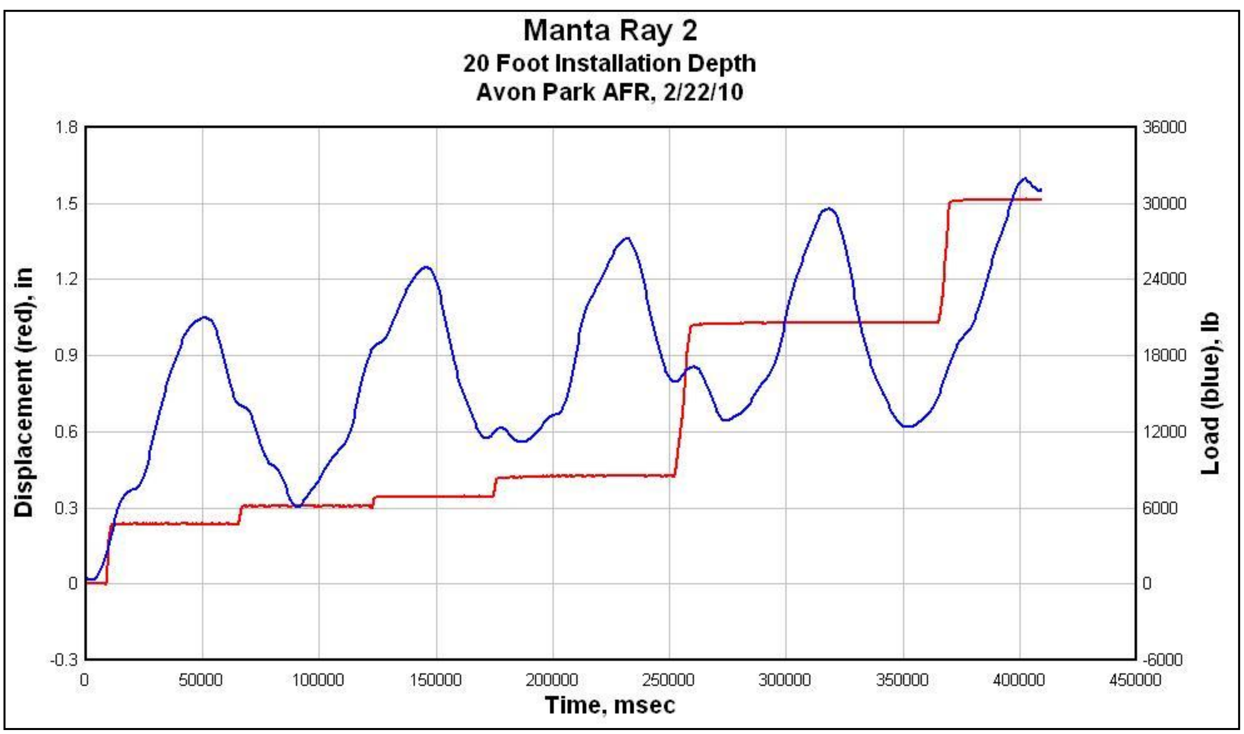

Fully Grouted 3
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
36000
30000
24000
18000
12000
6000
0
-6000
0
50000
100000
150000
200000
250000
300000
350000
400000
450000
500000

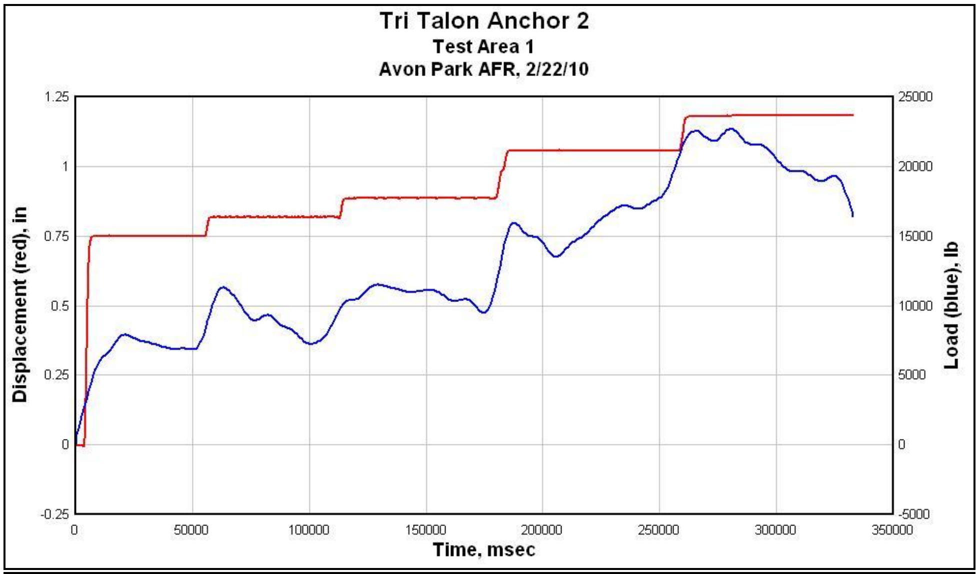

Partially Grouted 1
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1000psi
1500psi
2000psi
2500psi
3000psi

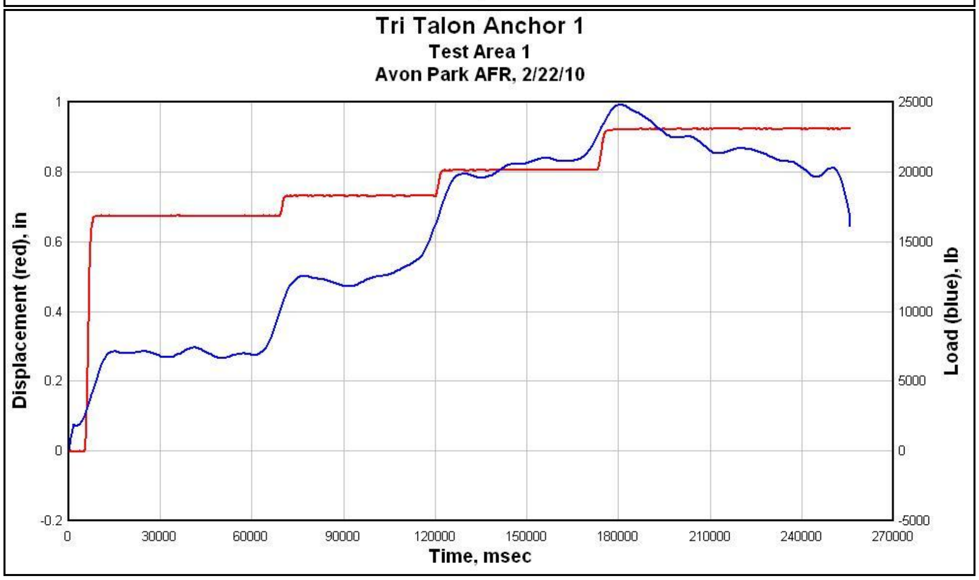

Partially Grouted 2
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1000psi
1500psi

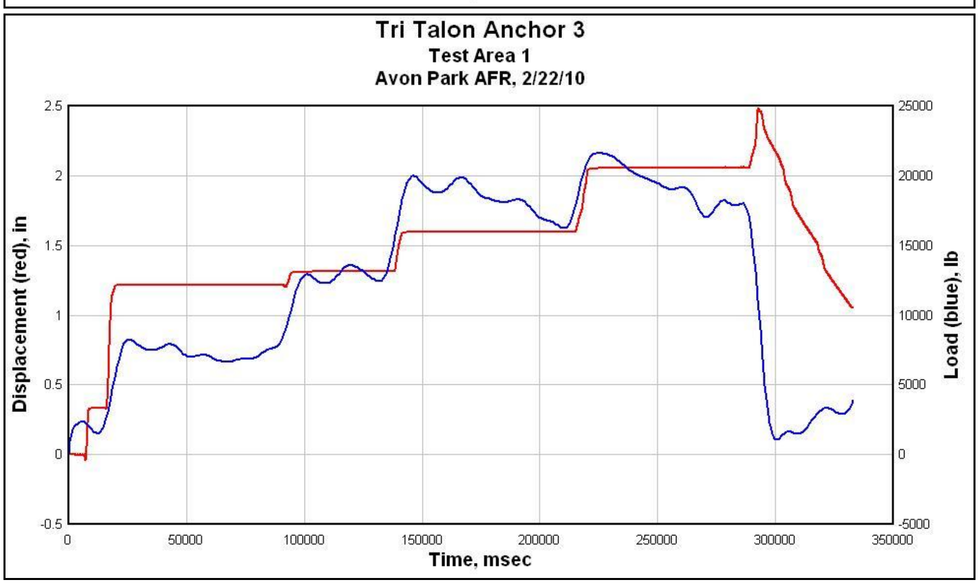

Partially Grouted 3
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1000psi
1500psi
2000psi
2500psi
3000psi

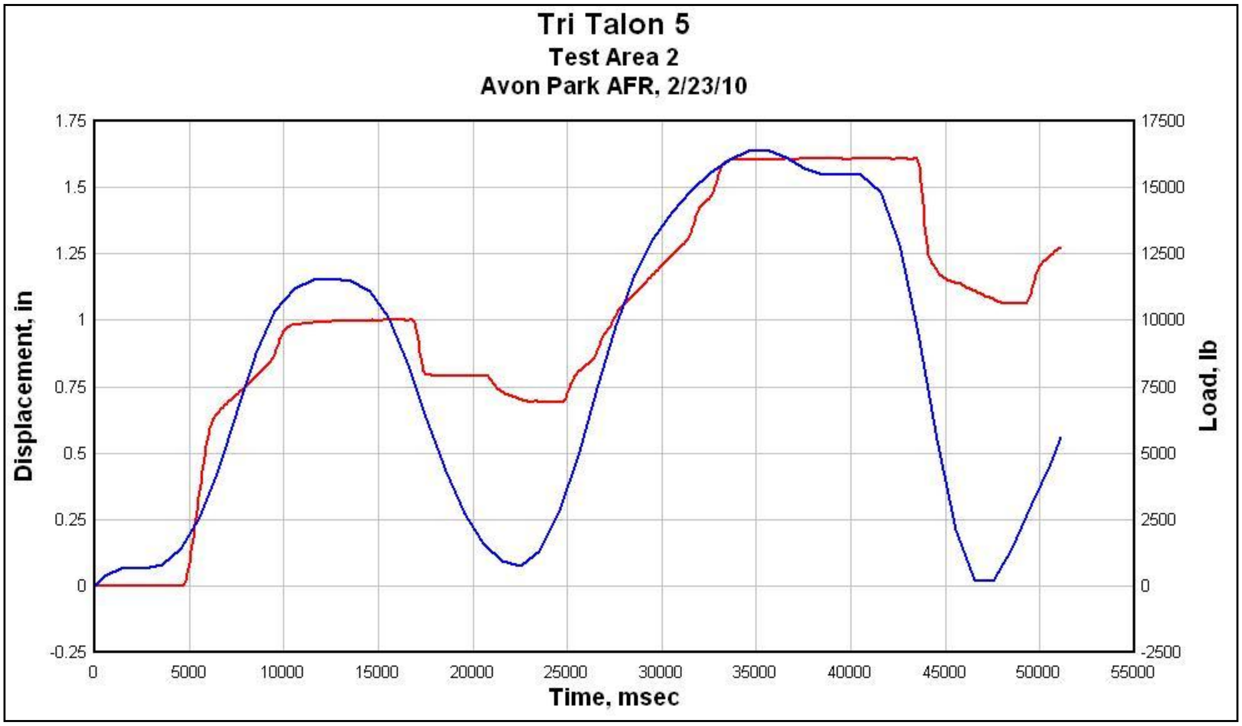

Manta Ray 1
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
30000
25000
20000
15000
10000
5000
0
-5000
0
50000
100000
150000
200000
250000
300000
350000
400000
450000
500000
550000

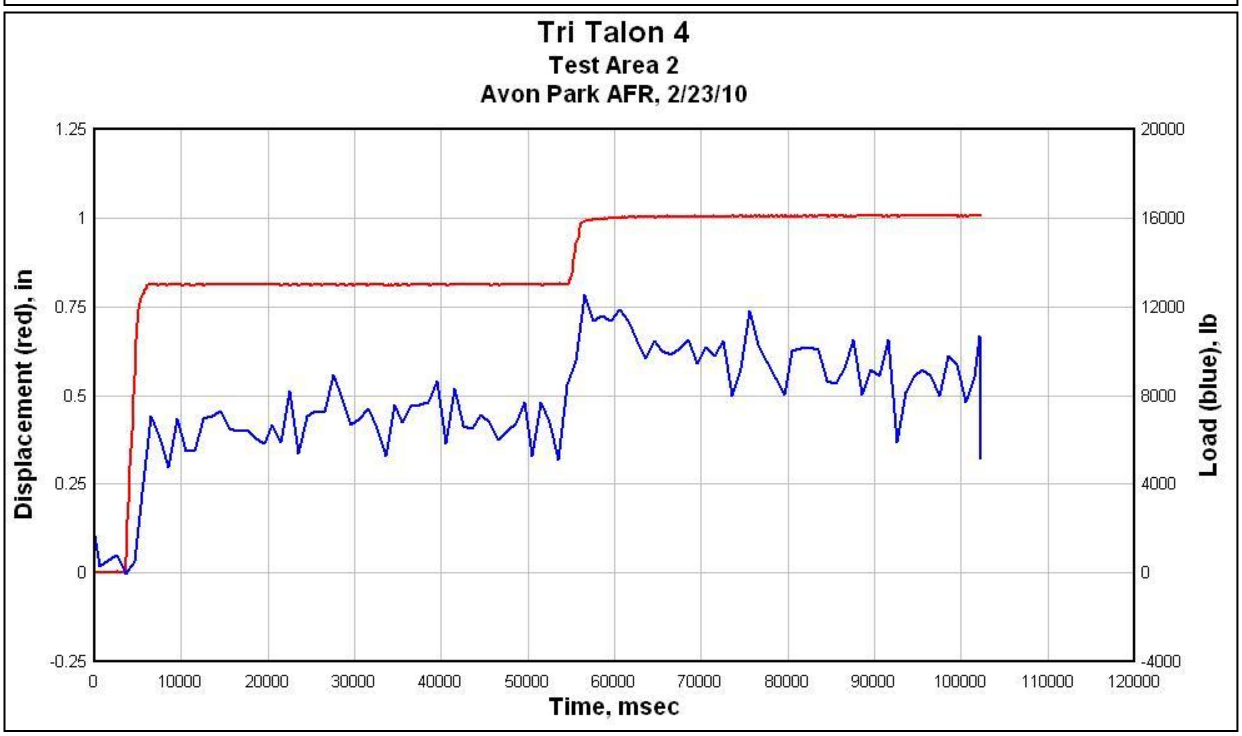

Manta Ray 2
20 Foot Installation Depth
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1.8
1.5
1.2
0.9
0.6
0.3
0
-0.3
36000
30000
24000
18000
12000
6000
0
-6000
0
50000
100000
150000
200000
250000
300000
350000
400000
450000

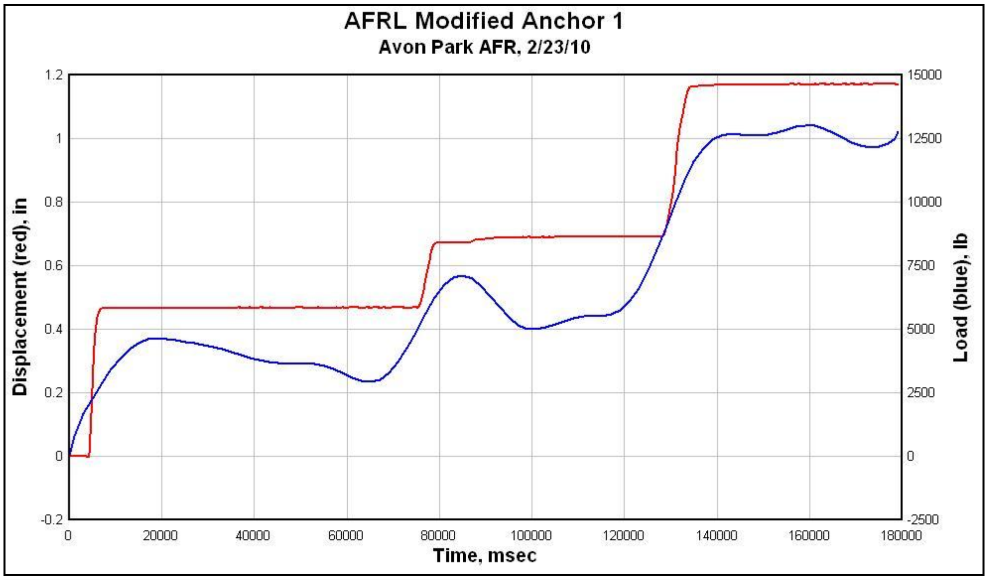

Tri Talon Anchor 2
Test Area 1
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1.25
1
0.75
0.5
0.25
0
-0.25
25000
20000
15000
10000
5000
0
-5000
0
50000
100000
150000
200000
250000
300000
350000

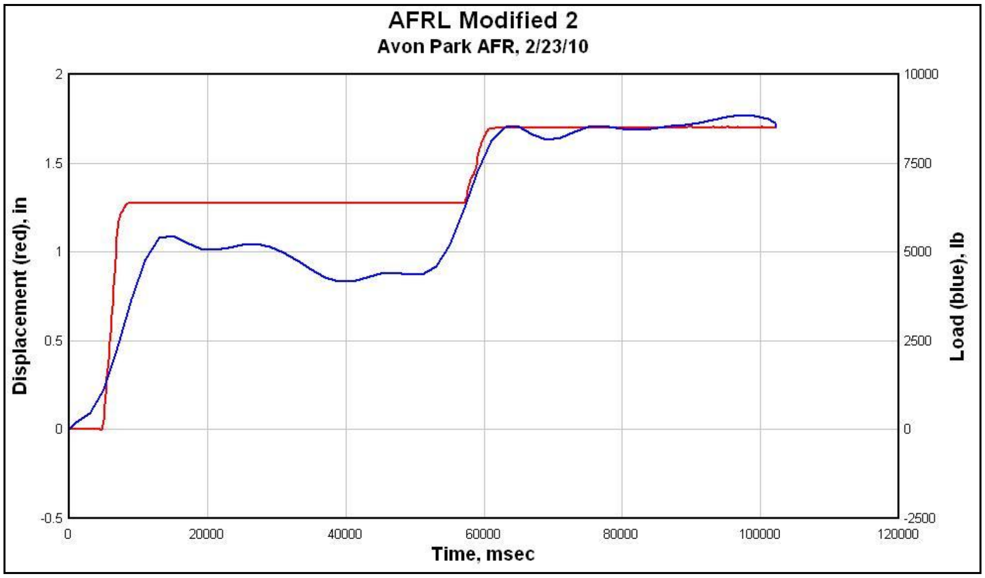

Tri Talon Anchor 1
Test Area 1
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
1
0.8
0.6
0.4
0.2
0
-0.2
25000
20000
15000
10000
5000
0
-5000
0
30000
60000
90000
120000
150000
180000
210000
240000
270000

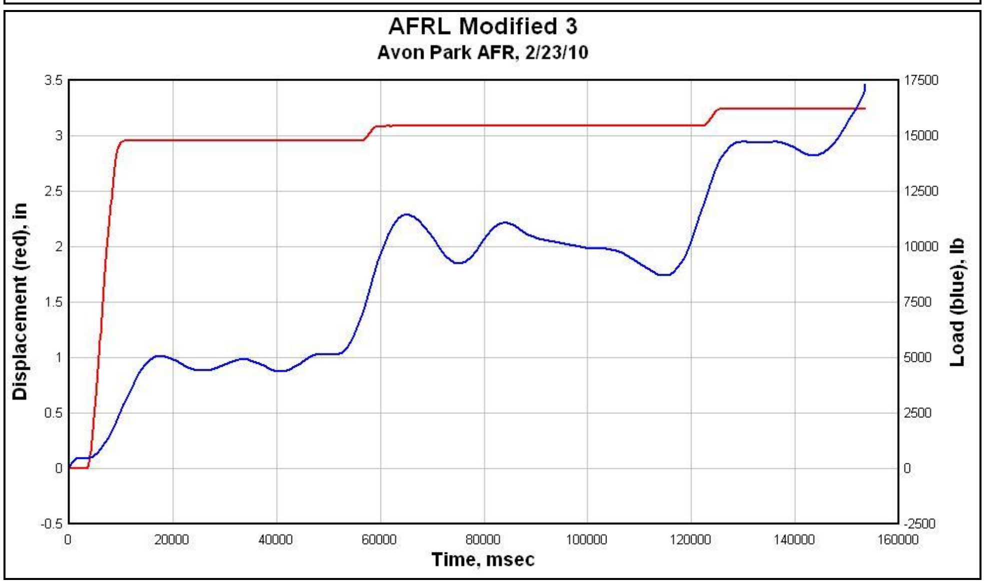

Tri Talon Anchor 3
Test Area 1
Avon Park AFR, 2/22/10
Displacement (red), in
Load (blue), lb
Time, msec
2.5
2
1.5
1
0.5
0
-0.5
25000
20000
15000
10000
5000
0
-5000
0
50000
100000
150000
200000
250000
300000
350000

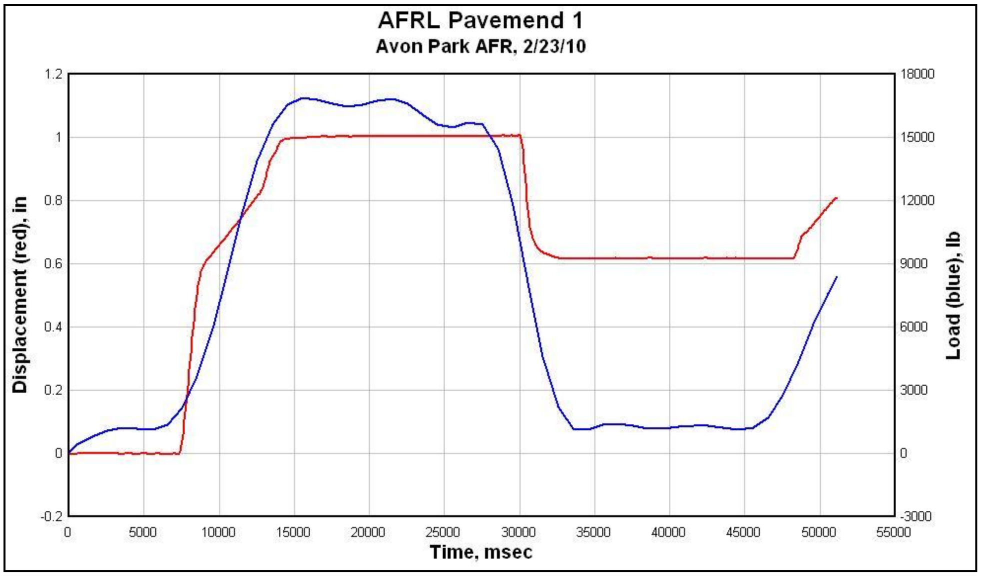

Tri Talon 5
Test Area 2
Avon Park AFR, 2/23/10
Displacement, in
Load, lb
Time, msec
1.75
1.5
1.25
1
0.75
0.5
0.25
0
-0.25
17500
15000
12500
10000
7500
5000
2500
0
-2500
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
50000
55000

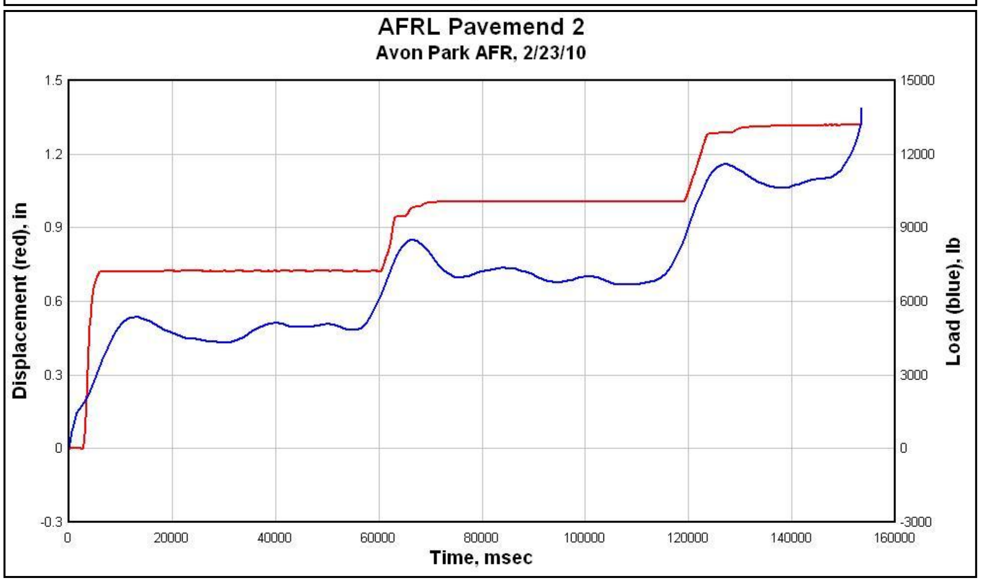

Tri Talon 4
Test Area 2
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec
1.25
1
0.75
0.5
0.25
0
-0.25
20000
16000
12000
8000
4000
0
-4000
0
10000
20000
30000
40000
50000
60000
70000
80000
90000
100000
110000
120000

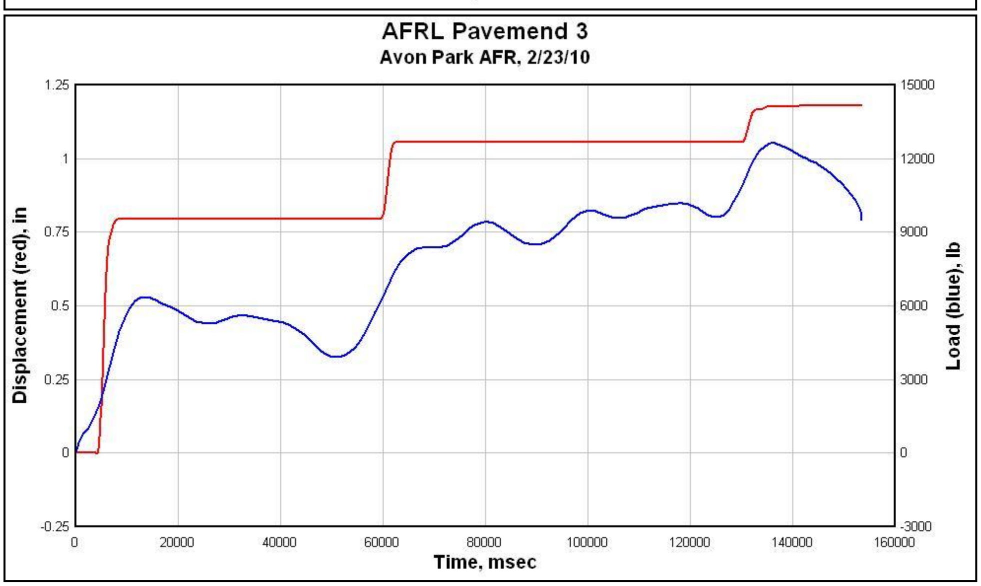

AFRL Modified Anchor 1
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec


AFRL Modified 2
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec


AFRL Modified 3
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec


AFRL Pavemend 1
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
18000
15000
12000
9000
6000
3000
0
-3000
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
50000
55000


AFRL Pavemend 2
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec
1.5
1.2
0.9
0.6
0.3
0
-0.3
15000
12000
9000
6000
3000
0
-3000
0
20000
40000
60000
80000
100000
120000
140000
160000


AFRL Pavemend 3
Avon Park AFR, 2/23/10
Displacement (red), in
Load (blue), lb
Time, msec
1.25
1
0.75
0.5
0.25
0
-0.25
15000
12000
9000
6000
3000
0
-3000
0
20000
40000
60000
80000
100000
120000
140000
160000

LIST OF SYMBOLS, ABBREVIATIONS AND ACRONYMS

AFB	Air Force Base
AFCESA	The Air Force Civil Engineer Support Agency
AFRL	Air Force Research Laboratory
AOS	aircraft operating surface
ERDC	U.S. Army Engineering Research and Development Center
FFM	folded fiberglass matting
ft	feet
Hz	hertz (= cycles/second)
in	inch
hp	horsepower
lb	pound
MR-SR	SR model Manta Ray earth anchor
mm	millimeter
PCC	Portland cement concrete
psi	pounds per square inch
SPT	standard penetration test
s	skin friction
t	tons